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**Published every Thursday.** Price 25c. per copy. Subscription, domestic, Central and South America, $5 per year. Canada, including duty, $7.50. Foreign $9 or 45 shillings. Entered as second-class matter April 11, 1917, at the Post Office at New York, N. Y., under the Act of March 3, 1879. **Printed in U. S. A.** Cable Address "MCGRAWHILL, New York." **Member A.B.P. Member A.B.C.**

MCGRAW-HILL PUBLISHING COMPANY, INC., 330 WEST 42D STREET, NEW YORK, N. Y.

Editorial and Publishing Offices: 330 West 42d St., New York; 520 North Michigan Ave., Chicago; 883 Mission St., San Francisco; Aldwych House, London, W. C. 2, England. Branch Offices: Washington; Philadelphia; Cleveland; Detroit; St. Louis; Boston; Greenville, S. C. James H. McGraw, Chairman of the Board; Malcolm Muir, President; James H. McGraw, Jr., Vice-President and Treasurer; Mason Britton, Vice-President; Edgar Kobak, Vice-President; H. C. Parmelee, Vice-President and Editorial Director; Harold W. McGraw, Vice-President; C. H. Thompson, Secretary

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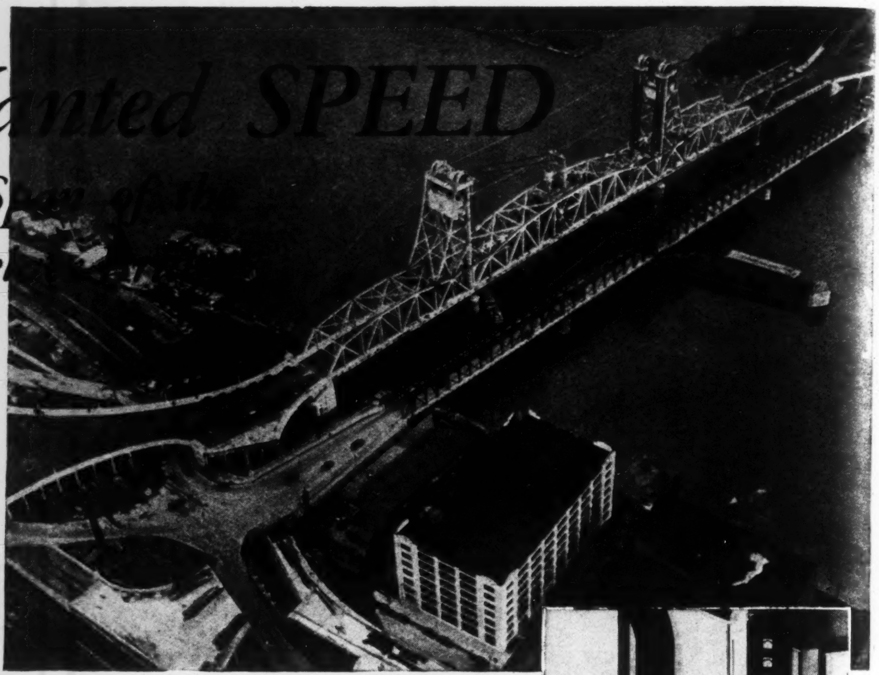
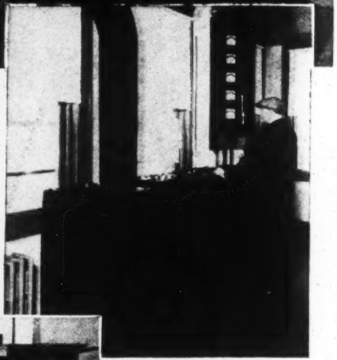


*The new vertical lift-span bridge across the Hudson at Albany, N. Y.*



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# Earthquake Design for Bank Buildings

## Four new branches of the Mitsui Bank in Japan illustrate the application of modern earthquake-proof design practices

By Paul F. Pape
*Weiskopf & Pickworth, Consulting Engineers, New York, N. Y.*

FOUR bank buildings, one each at Yokohama and Nagoya, and two in Osaka, Japan, embodying the latest ideas on earthquake-proof design and construction, were completed last year for the Mitsui Bank of Tokyo. These branch banks were begun in 1929, shortly before the completion of the main Mitsui Bank building in Tokyo (*ENR,* June 23, 1927, p. 1,010), which was one of the first structures to be built after the 1923 earthquake and was designed on a modern earthquake-proof basis. In both foundations and superstructures the branch banks follow the most advanced principles of earthquake-proof design, while local conditions made the several buildings interestingly different.

The dominant factors governing the architectural and structural design of these buildings (Fig. 1) were function, appearance and strength. The function of each building was to provide vaults in the basements, a banking room on the main floor and executive offices on the floors above. An appearance of sturdiness, endurance and mass was given to the buildings by the facade design. The strength of the building was developed by proportioning and interconnecting its elements (foundation, structural frame, retaining walls and curtain walls) so that the whole would move and act as a unit to resist earthquake shock in addition to the vertical loads. A seismic coefficient of 10 per cent of the total weight above any plane, and acting horizontally in any direction, was used in the design, as required by the Japanese building law. Since the vertical component of the earthquake shock is comparatively small, it was neglected in these designs.

An examination of each site for soil-bearing capacity and relative elevation of the water table was first made. A fine sand and gravel stratum that showed practically no settlement under a test load of 4 tons per sq.ft. was found on all sites except Kawaguchi, in Osaka. The water table was below the foundation mat at Semba (Osaka) and Nagoya

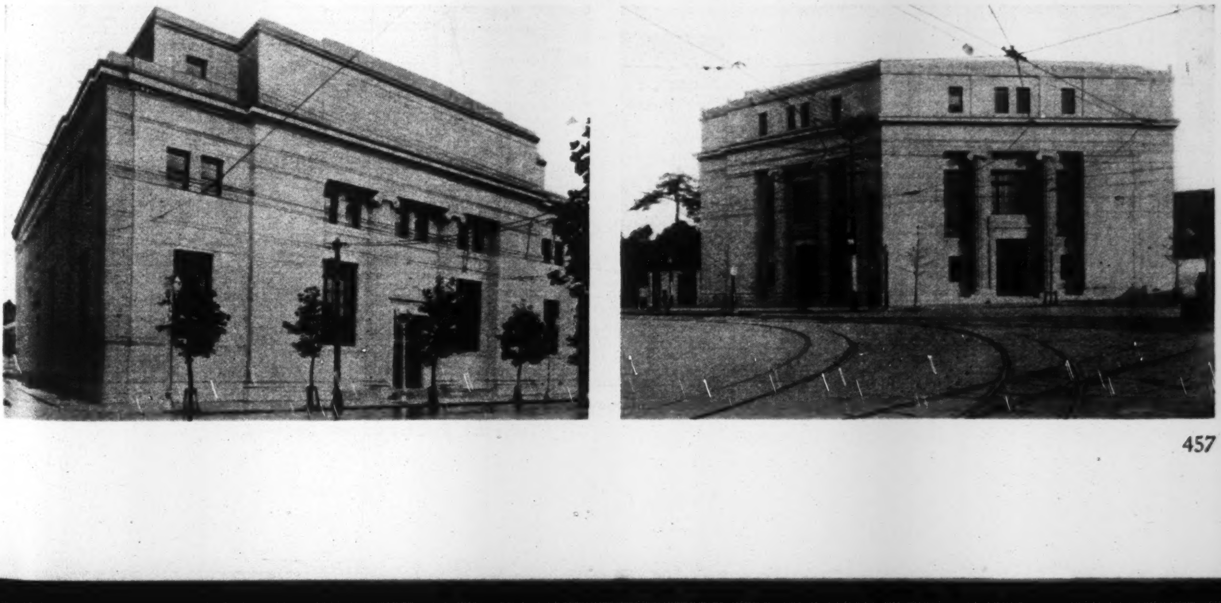


Fig. 1—Two Japanese banks that are designed to resist strong earthquakes—Semba bank in Osaka at left, and bank at Nagoya at right. Continuous foundation mats, structural-steel frames, reinforced-concrete floors, walls and partitions, all tied together to act as a unit, comprise the design basis.

and above at Kawaguchi (Osaka) and Yokohama. On the Kawaguchi site piles were required. The strata on the Kawaguchi site consisted of 3 ft. of loam, 10 ft. of sandy clay and 8 ft. of fine sand over a 60-ft. layer of elastic clay. Short piles were tested on the strata above the elastic clay layer, but these proved insufficient for the loads to be handled. Therefore, it was necessary to use wood piles 90 ft. long, consisting of a lower 60-ft. pile and an upper 30-ft. pile spliced with a long metal cylinder. These piles were driven through the elastic clay layer to a 21-ft. sand layer overlaying thin layers of clay, sand and gravel. These piles tested 30 tons each.

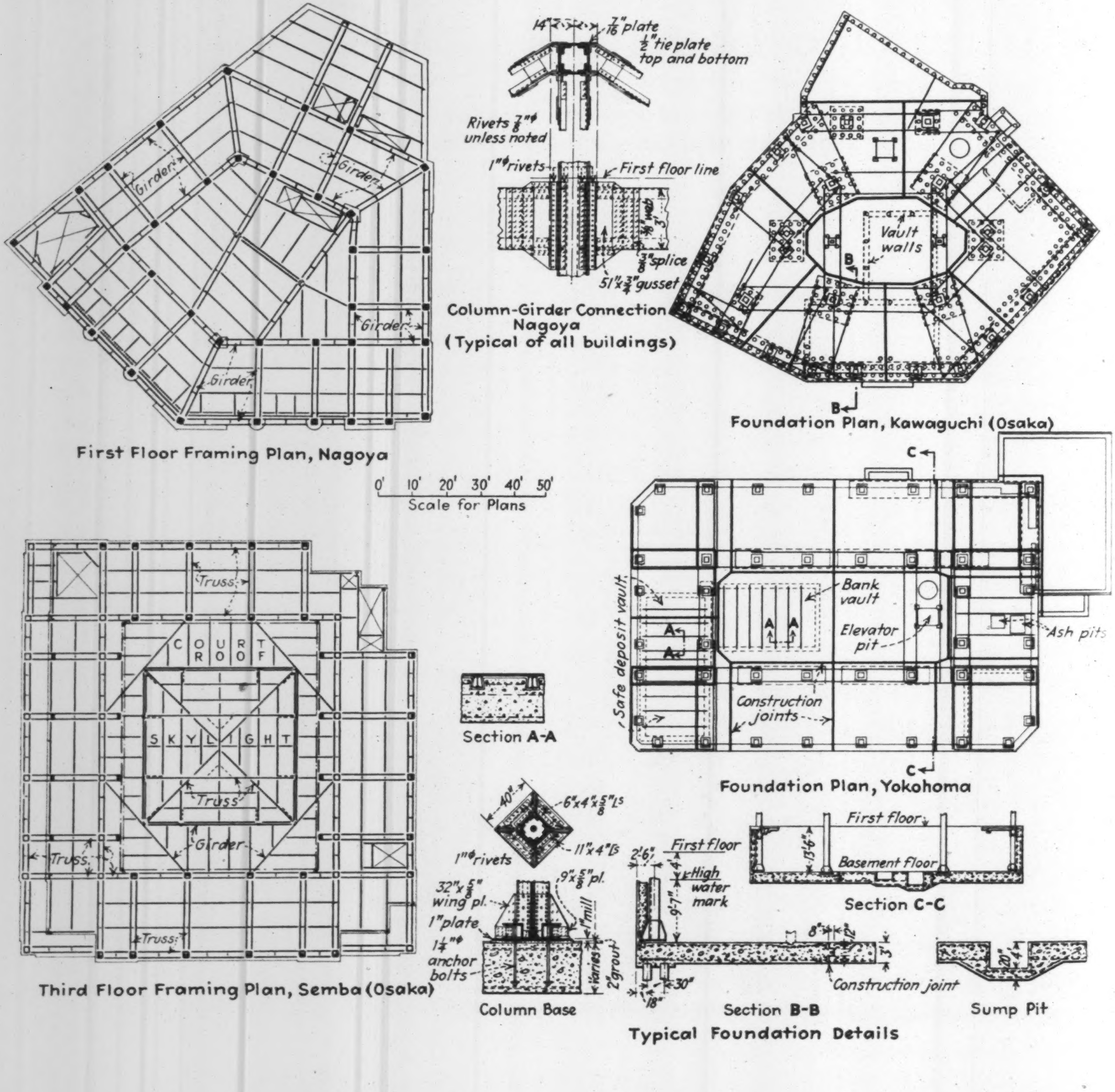
The foundation mat designs for the four jobs were similar, the chief difference resulting from the necessary provision for water pressure at two of the sites.

### Foundations above water table

The design of the foundation mats at Semba and Nagoya took into consideration the spreading of vertical loads to the soil, with an allowable maximum of 2 tons per sq.ft., the necessary resistance to shock and the provision of adequate joints and ties between mat sections. By tying together all footings any uneven settlement of individual footings, with the consequent racking and disruption of the structural frame, was overcome. Cantilever spread footings within the mat were used to distribute the column and wall loads and confine them within the building lines. The exterior column and wall loads were spread parallel to the building line by a 5-ft. band of rods and were cantilevered at right angles onto a 10-ft. bearing band. The exterior moment of these cantilevers was balanced by an uplift on the interior column. In spreading the interior column load a 5-ft. band parallel to the building line was used whenever the column loads were large. Thus, the usual foundation reinforcing consisted of a closed rectangle or polygon of bands under the interior and exterior columns, the two sets of bands being tied together by other bands of rods (at right angles to them). A regularity of layout was possible because symmetry about the center mass of the building was fairly well attained on these buildings.

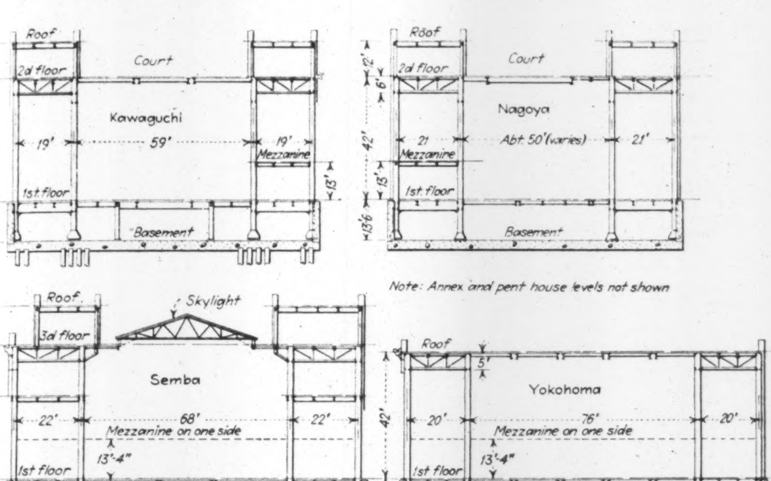
Two layers of reinforcing rods, one 4 in. below the top of the mat and the other 4 in. above the bottom, were used in each of the 3-ft. thick mats. Where no reinforcement was needed for beam action a minimum amount of 0.5 sq.in. of steel per foot of width of slab was used in both directions in both top and bottom layers. In compliance with the

**Fig. 2—Typical foundation and superstructure details of the Japanese bank buildings, designed to resist earthquakes.**


First Floor Framing Plan, Nagoya
Girder
Girder
Girder
Girder
14"
7/16" plate
1/2" tie plate top and bottom
Rivets 7/8"φ unless noted
1"φ rivets
First floor line
3/8" splice
5 1/2"x3/4" gusset
Column-Girder Connection Nagoya (Typical of all buildings)
Vault walls
B
B
Foundation Plan, Kawaguchi (Osaka)
0' 10' 20' 30' 40' 50'
Scale for Plans
Truss
COURT ROOF
SKYLIGHT
Truss
Girder
Truss
Truss
Third Floor Framing Plan, Semba (Osaka)
C
Bank vault
Elevator pit
Ash pits
Safe deposit vault
A
A
A
A
Construction joints
C
Foundation Plan, Yokohoma
Section A-A
40"
6"x4"x5/8" Ls
11"x4" Ls
1"φ rivets
9"x5/8" pl.
32"x5/8" wing pl.
1" plate
1 1/4"φ anchor bolts
1" mill
Varies
2" grout
Column Base
2'6"
First floor
High water mark
9'-7"
8"
12"
3'
30"
18"
Construction joint
Section B-B
First floor
Basement floor
13'-6"
Section C-C
20"
4"
Sump Pit
Typical Foundation Details

Japanese building law all reinforcement was anchored at the ends by hooks or bends.

The foundation mat construction joints (Fig. 2) were designed to provide as much continuity as possible. Their positions were determined by the relative location of the spread footings and the maximum volume of the daily pour. Where depressions, such as sump and ash pits, occurred in the foundation mat the continuity was also maintained as nearly as possible, and the projecting bumps (Fig. 2) were smoothed out on the lower side by a 2:1 beveled projection, which will minimize undue friction when shock moves the mat over the soil.


Roof
Court
2d floor
Kawaguchi
19'
59'
19'
Mezzanine
1st floor
Basement
12'
42'
13'
13'6"
Roof
Court
2d floor
Nagoya
21
Abt. 50'(varies)
21'
Mezzanine
1st floor
Basement
12'
6'
42'
13'
13'6
Roof.
Skylight
3d floor
Semba
22'
68'
22'
Mezzanine on one side
13'-4"
1st floor
3'
Basement
13'
17'
28'
13'6"
Roof
5'
Yokohoma
20'
76'
20'
Mezzanine on one side
13'-4"
1st floor
3'
Basement
42'
13'6"

Note: Annex and pent house levels not shown

**Fig. 3—Cross-sections of four bank buildings in Japan, in which high banking rooms had to be accommodated. Framing is of structural steel, with heavy floor girders or trusses and deep connections. Concrete walls and partitions are used.**

### Foundations below the water table

The foundation mat design for Kawaguchi (Osaka) and Yokohama included provision for upward water pressure, for spreading vertical loads to the soil or to piles (with an allowable maximum of 2 tons per sq.ft. and 18 tons per pile, respectively, for Yokohama and Kawaguchi), for taking care of shock and for adequate joints and ties, between the mat sections, that would make the foundation watertight and continuous. These foundation plans, shown in Fig. 2, are in all general features similar to those for Nagoya and Semba. The retaining wall around the edge of the mat and the closed rectangle or polygon of bands under the interior columns were used as resisting members against uplift. The column and wall loads were spread as at Semba and Nagoya.

The safe-deposit and security vaults in the basement consisted of concrete walls and roof set on and keyed to the foundation mat. These were heavily reinforced with layers of expanded metal and protected on the inside by steel lining laminated with a layer of torch-resisting metal between two layers of open-hearth steel.

Special vault-floor construction, as an extra precaution against leakage through the foundation mat, was developed (Fig. 2, Sec. *AA*). The I-beams were sunk in the slots in the foundation mat with their ends anchored, and bearing under and upward against the vault walls. Floor plates were tap-bolted to the beams and waterproofed.

The retaining wall around each of the four buildings was made 2 ft. thick and was jointed vertically and horizontally. The vertical joint was made midway between columns. The horizontal joint was made at such an elevation as to allow erection and riveting of the exterior columns and the first-floor framing. The retaining wall was doweled to the foundation mat, to the first-floor slab and to the curtain wall above.

During construction a cross-lot system of bracing for the sheetpiling driven around the building lines was used where water pressure was present. In other cases a braced-bent system radiating from a center foundation was used. To avoid the effects of a severe shake that might shut down the electrical and water-supply systems during the pouring of the mat or the period in which the site was being drained, extra water-supply tanks and gas-engine alternates for pumping as well as for mixing concrete were provided.

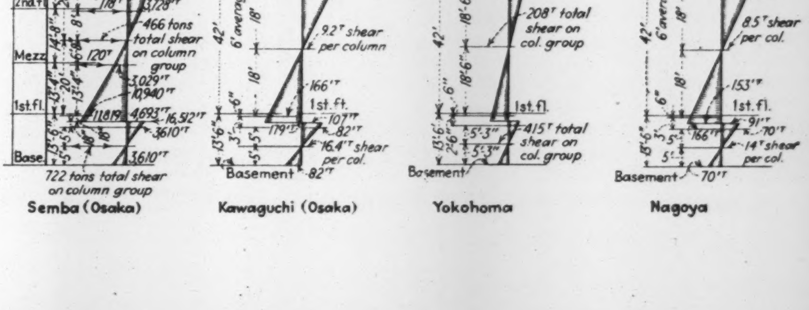
### Superstructure design

To meet architectural requirements as well as to take tensile stress induced by flexual action and the reversal of stresses resulting from alternating earthquake vibrations, structural-steel frames were adopted as the best type of construction for the bank buildings.

In the design it was necessary to consider the buildings from a broad viewpoint in deciding how this resisting unit should be proportioned to take the vertical loads and the earthquake shock. Each member and its joints were proportioned for strength and to do their share of the resisting work. The method of proportioning for shock was governed to a large extent by the shape and continuity of the building. On the irregular-shaped buildings the horizontal shear was assumed as acting from any direction and as being taken equally by each column (Fig. 4). The resisting strength figured for each floor member in each direction was governed by the number of members framing into the column in that direction, since the summation of the resisting strength of the members must be equal to or larger than the column's resisting strength. The column material was designed for the horizontal thrust diagonal to the normal axis, which produced maximum stress.

On the rectangular-shaped buildings it was evident that a degree of standardization could be carried out in the design of the members and their joints. With this in view, a shock moment diagram for the entire group of columns in each building was made, and the members and their connections were proportioned in each direction in relation to the sum total in that direction.

**Fig. 4—Moment diagrams upon which earthquake-proof designs of Japanese banks were based. Note that for the irregular-shaped buildings (Nagoya and Kawaguchi) the diagram is for an individual column, while for the regular-shaped buildings a group of columns is the unit.**


104 tons total shear on column group
Roof
676"T
624"T
3rd.fl.
624"T
8784"T
884"T
7900"T
1184"T
2nd.fl.
178T
3728"T
466 tons total shear on column group
Mezz.
120T
3029"T
10940"T
1st.fl.
11819
4693"T
16512"T
3610"T
Base.
3610"T
722 tons total shear on column group
Semba (Osaka)
16"T
Roof
15"T
2.8T shear per col.
2nd fl.
15"T
24"T
19.3"T
166"T
6' average
9.2T shear per column
166"T
1st. ft.
107"T
179"T
82"T
16.4"T shear per col.
Basement
82"T
Kawaguchi (Osaka)
Roof
208T total shear on col. group
1st.fl.
415T total shear on col. group
5'-3"
Basement
Yokohoma
16"T
Roof
15"T
2.7T shear per col.
15"T
2nd.fl.
178.5"T
23"T
153"T
6' average
8.5T shear per col.
153"T
1st.fl.
91"T
166"T
70"T
14T shear per col.
Basement
70"T
Nagoya

All typical girder connections were made alike, as were all truss shock members. The columns were then proportioned for shear and moment, according to the number of shock members framing into them in that given direction. The critical condition of the girders was when the direction of the shock was parallel to them. The critical condition of the columns was when the shock was applied on a diagonal. Shear diagrams for each of the buildings are shown in Fig. 4.

The steel frame was incased in concrete and was made as rigid as possible, so that the whole structure would move and act as a self-contained resisting unit, comparable to a ship afloat. Following the lines laid down by the Japanese law, all buildings were kept under 100 ft. in height. Reinforced-concrete floor slabs were figured to take the horizontal shear of the earthquake and distribute it to each column. The floor construction consisted of a 4-in. reinforced-concrete slab, a 4½-in. lava fill and a 1-in. cement finish. Live loads of 100, 75 and 50 lb. per sq.ft. were used on the banking-room floor, office floors and roofs, respectively.

*Columns*—A four-angle column section, laced or plated as required, was adopted because of its strength when bending action from any direction was considered, and because of the ease of keeping it and its connections within the architectural requirements. With this section it was convenient to make double parallel connections on each face and to increase the section and therefore the resisting strength of the column at points of high moment, such as at girder and truss connections.

The columns and column bases were designed for shock moment, uplift and vertical load. The bases spread the load uniformly to the foundation mat by means of wing plates attached to each face of the column section. Channel members between the wing plates served to take the anchor bolts as well as to aid in spreading the load.

*Girders and Trusses*—There was a large variation between the shock moments at the first-floor level and the roof level, and the members designed to resist them were plate girders at the first floors, trusses above the banking rooms and girder beams for the roofs and intermediate floors. These members met the structural and architectural requirements, and also furnished adequate bracing and stiffening where needed. The high open banking rooms greatly increased the moments that had to be resisted.

The first-floor girders, which required the largest resisting moments, consisted of a solid web plate and single chord angles forming a channel-shaped section. The connection rivets on the column are in double shear and aid in cutting down the size of connection required. Japanese standard rolled sections were used throughout and were designed for a fiber stress of 16,000 lb. per sq.in.

The banking-room ceiling trusses were from 5 to 8 ft. deep, dependent upon conditions, and consisted of single angle members. On the Semba building the roof columns and the court roof were supported by cantilever trusses. All girders and trusses were braced laterally by the floor framing, and each pair was tied together at intervals by plate separators.

*Walls*—The exterior walls were finished with Japanese granite backed up with reinforced concrete. The facing was anchored and bonded to the backing, which had a double layer of reinforcement with rods at 1-ft. intervals both ways. This backing was also anchored and doweled to the floor slabs and the steel columns at frequent intervals. Large window openings existed in each bay of the exterior walls on two or more sides of the buildings. However, since the exterior walls were not figured as resisting members, no attempt was made to design heavy lintel beams or vertical column wings to compensate for the loss of strength. Fixed interior partitions consisted of reinforced concrete 6 in. thick.

Trowbridge and Livingston, New York City, were the architects on the four Mitsui branch banks. Weiskopf & Pickworth, of New York City, were the consulting engineers. The writer worked on the designs in New York and later spent seventeen months in Japan in charge of construction and structural detail. The general contract work was done by Japanese contractors, as follows: Yokohama, Shimizu Gumi; Semba in Osaka, and Nagoya, Tanaka Kumetan; and Kawaguchi in Osaka, Obayashi Gumi. J. Ishii, in charge of the Nagoya and Semba projects for Tanaka Kumetan, materially aided the writer in organizing the work.

---

# Salinity in Sacramento River Delta Area Reviewed

INVASION of saline water in the upper reaches of San Francisco Bay and the deltas of the Sacramento and San Joaquin rivers, with its serious effect on rich agricultural areas and industrial development, is a natural, annual phenomenon that has been accentuated during the past decade by (1) deficient precipitation and (2) increased consumptive use on the rivers, according to a report prepared by the California division of water resources under the direction of Edward Hyatt, state engineer. About 1916 the subject first aroused interest, and from 1920 a comprehensive study has been in progress on the saline encroachment, its consequences and methods of prevention. Local agricultural interests and the industrial organizations situated on the shores of upper San Francisco Bay were active in sponsoring the collection of these data, and the state's interest resulted, in part, from the inter-relation of this problem with the proposed state plan for the complete development of its water resources.

Within the delta area, about 20 miles wide by 50 miles long, there are 421,000 acres of highly productive agricultural land that has been gradually reclaimed from overflow channels at great cost during a period of 75 years. The reclaimed area, according to the report, contains about all the land within the delta feasible of reclamation. For the year 1929, 350,000 acres of this area produced a crop valued at $30,000,000, with principal commodities of asparagus, corn, potatoes, sugar beets, beans, celery and fruit. The taxable wealth of the delta area is about $45,000,000. The industries located along the shores of the upper bay are vitally interested in the question of salinity control as a result of using 7 m.g.d. of fresh water (1929) for boiler and the process purposes. These industries have no storage facilities, depending on direct diversion, and they have serious difficulties in obtaining fresh water during the summer periods of saline invasion. The conclusions reached, as outlined below, have curtailed expansion and increased costs because of the introduction of substitute underground supplies.

1. The invasion of salinity into Suisun Bay (upper enlargement of San Francisco Bay), as far as the Sacramento-San Joaquin delta, is a natural phenomenon which, in varying degree, has occurred each year as far back as historical records reveal.

2. The abnormal degree and extent of the saline encroachment since 1917 has been due chiefly to: (1) Subnormal precipitation and runoff with the reduction in the streamflow naturally available in the delta; and (2) increased upstream diversion for irrigation and storage on the river system which has probably doubled the degree of salinity in this area.

3. During several years since 1920 the inflow into the delta during the summer months has been insufficient to meet the consumptive demands within the area, which vary from a winter minimum of 400 sec.ft. to a summer irrigation peak of 3,700 sec.-ft.

4. Tidal action has been found to have a direct effect on the salinity encroachment, as the rise and fall of the tide have a pulsating action exerting a

tendency to push upstream and mix with the fresh water. The streamflow opposes this action, tending to push the saline water downstream; and the relative magnitude of these opposite and opposing forces governs the actual occurrence and advance of salinity.

5. To protect the delta area from saline invasion, it would be necessary: (1) To provide sufficient water flowing into the delta to satisfy consumptive demands, together with natural losses; and (2) to provide additional flow sufficient to repel tidal action and the invasion of salinity.

6. The prevention of saline encroachment in a harmful degree would require a 3,300-sec.-ft. net flow in the combined rivers entering the upper bay. With this flow maintained past the city of Antioch, the maximum degree of salinity would be limited to 100 parts of chlorine per 100,000 at that point, and to 10 parts per 100,000 or less within the principal agricultural areas of the region.

7. The fresh-water supply required for effective control and consumptive use within the delta must provide a flow varying from about 4,000 sec.-ft. in winter to a maximum of 7,000 sec.-ft. in mid-summer.

8. Streamflow into the delta area during the past ten years has been insufficient during summer months to provide the required control for salinity. Supplemental water supplies would be necessary to meet the deficiency. This supplemental water supply required for salinity control by streamflow could be developed and provided by the mountain storage reservoirs now proposed in the state water plan.

The suggested solution of this problem by means of a physical barrier to control the entrance of saline water is not discussed in the present report, since its physical and economic aspects have been covered by previous state reports. The study was carried out by the department of public works, under the general supervision of Edward Hyatt, state engineer. The bulletin was prepared under the direction of A. D. Edmonston, deputy state engineer, by Raymond Matthew, hydraulic engineer.

---

# New Type of Reinforcing Bar Develops High Bond Stress

By Chesley J. Posey
*Instructor in Mechanics and Hydraulics, University of Iowa, Iowa City, Iowa*

DURING the progress of an investigation conducted by the writer in the engineering laboratories at the University of Iowa, a new type of reinforcing bar has been evolved that will develop considerably higher bond stress, with less slip, than previous types of deformed bars. It differs from existing types in that the deformations are small and so closely spaced that they might properly be called crenulations. Fig. 1 illustrates two variations of the crenulated type of bar that were tested, also an ordinary deformed bar for comparison. The accompanying table of test results gives the comparative values of bond stress at different amounts of slip for 13$\frac{3}{4}$-in. embedments of the $\frac{5}{8}$-in.-diameter bars cast in concrete having an ultimate compressive strength of 2,000 lb. per sq.in. The slip measurements were made at a point 1$\frac{7}{8}$ in. from the loaded end of the embedment and should not be compared with those of tests in which slip measurements are made at the free end of the bars. Since slips exceeding 0.01 in. are not generally regarded as permissible, the bond stress for a slip of 0.02 (given in the table) is not particularly significant. The other results show that at slips of 0.01 in. or less the crenulated bars developed stresses about 50 per cent higher than did the ordinary deformed bars. The tests also indicated that the crenulated bars are less likely to split the concrete than ordinary deformed bars.

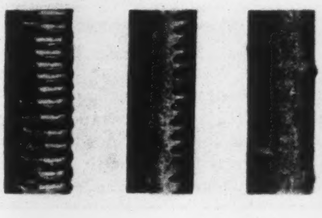


**Fig. 1—Two variations of a new type of reinforcing bar in which small, closely spaced notches or crenulations are used to develop a higher bond stress than the usual deformed bar, an example of which is shown at the right.**

The theory of the crenulated type of bar rests on securing a better balance between the shearing area and the bearing area presented to the concrete. Fig. 2 represents a longitudinal section through a deformed bar. The resistance to slipping is increased by the presence of the projections. To obtain maximum resistance, the projections should be spaced so that the shearing strength of the concrete along section *AB* will equal its bearing strength on the edge of the projection *AC*, which in turn should equal the shearing strength of the steel along section *DE*. It is also important that the angle of inclination of the bearing surface *AC* to the axis of the bar should be sufficient to prevent wedging action from splitting the concrete. With nearly all existing types of deformed bars, the shearing area along *AB* is too great in proportion to the bearing area on the edge of the projection *AC*. Better proportions in this respect are provided by twisted square bars. but they have the defect that the surface in bearing makes too small an angle with the axis of the bar. The two requirements seem to conflict if the area of the bar is not to vary along its length, which is an important practical consideration. The difficulty can be overcome by making the projections quite small compared to the diameter of the bar. In the absence of accurate information as to the relative strength of the concrete in inclosed bearing and shear, the best dimensions for the small deformations or crenulations cannot be determined. However, bars that only roughly approximate a proper balance give bond stresses that exceed by a considerable margin those obtained for ordinary deformed bars, as is evident from the test data.

BOND STRESSES DEVELOPED BY THE BARS OF FIG. 2

| Type of Bar (See Fig. 2) | Average Bond Stress When Slip = .005 In. | | Average Bond Stress When Slip = .01 In. | | Average Bond Stress When Slip = .02 In. | |
|---|---|---|---|---|---|---|
| | Lb. per Sq.In. | Relative to Bar *C* | Lb. per Sq.In. | Relative to Bar *C* | Lb. per Sq.In. | Relative to Bar *C* |
| Left Bar | 417 | 1.59 | 556 | 1.53 | 658 | 1.48 |
| Center Bar | 442 | 1.69 | 536 | 1.47 | 607 | 1.31 |
| Right Bar | 262 | 1.00 | 364 | 1.00 | 463 | 1.00 |

The tests upon which this article is based formed part of an experimental investigation of large-radius hooks and straight embedments as anchorages for reinforcing bars. Experimental work was carried on in 1930, '31 and '32. The tests will be fully described in *Engineering Series Bulletin No. 3*, to be published by the State University of Iowa.

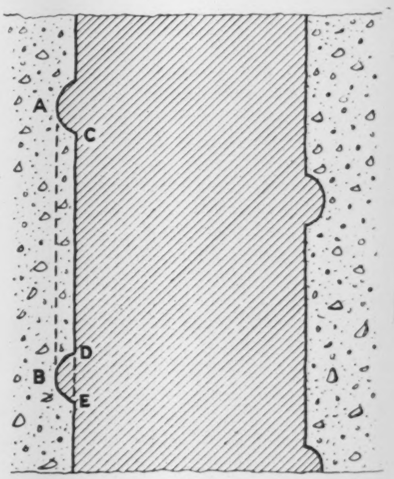


**Fig. 2—Section through a deformed bar embedded in concrete, showing the shear and bearing areas that need to be balanced for optimum bond-stress development.**

# Modern Concrete Design Exemplified in Brooklyn Apartments

**Flagg Court Apartments benefit from thin two-way slab floors and unplastered ceilings—Columns placed in inconspicuous corners and closets, and beams confined to partitions—Frame of unusually slender proportions—Straight reinforcing bars of rail steel used**

THE NEW nine-story Flagg Court Apartments in Brooklyn, N. Y., represent an attempt on the part of the designers to take full advantage of all the properties of reinforced concrete as they may be applied to housing construction. Both design procedure and the actual arrangement of the structural frame are notable. The design was based on the elastic theory (moments and shears determined by application of the theory of statically indeterminate structures), with no recourse to conventional moment coefficients. A frame of unusually slender proportions resulted. Thin slabs were used, reinforced in two directions with unbent rail-steel reinforcing. Plywood forms produced a ceiling surface satisfactory for painting and requiring no plaster. The column-and-girder system is arranged so that columns occur in closets or inconspicuous locations, flush with partitions on the room side. Beams are permitted to project into rooms only at partitions. In construction an unusual refinement of form building and installation and of reinforcing-bar placement was required and secured. As a result of careful concrete mix design, it was possible to strip forms at an early date, thus speeding up the work.

The building recently completed includes four units of an eventual eight-unit layout inclosing a central court to be occupied by two swimming pools (Fig. 2). Each unit covers a plot about 100 ft. square and, except for connections with other units at roof and basement, is a complete building in itself. The building is fireproof throughout—concrete frame and floors, 2-in. cement mortar partitions on steel mesh, and inclosure walls of brick with hollow tile backing. Roofs are designed as playgrounds. The project is in the Bay Ridge section of Brooklyn overlooking New York harbor.

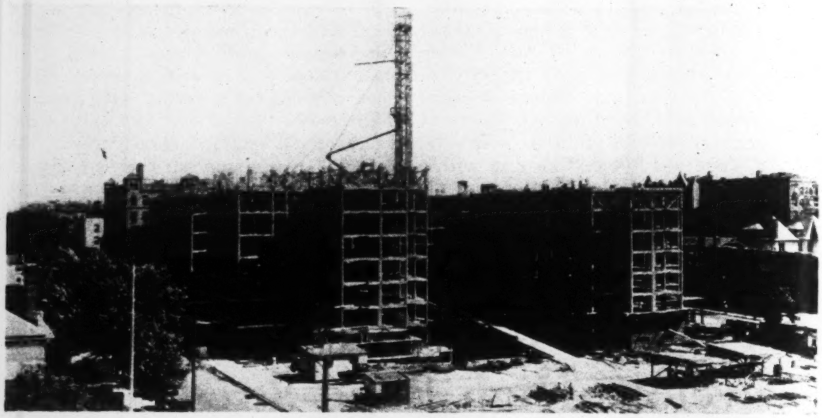


Fig. 1—The concrete frame of the Flagg Court Apartments in Brooklyn, in whose design and construction the most modern concepts and practices found application.

At the beginning of the work the architect submitted a floor layout to the designers and listed a number of basic requirements. Among these were the following: The frame must conform to the architectural layout and the proposed facade treatment; concrete forming the ceilings and all exposed surfaces of beams was to be sufficiently smooth to match, when painted, a plastered surface; the floor to floor dimensions were fixed at 8 ft. 6½ in. and the headroom in all parts of the room at 8 ft., 1 in. allowance being required for a ⅞-in. oak floor glued to the finished concrete slabs with hot asphalt; beams were allowed to project below the ceilings along the walls and partitions but had to clear the window heads at the exterior walls, which were 10½ in. below the top of the concrete slabs, and the door heads at the exterior partitions, which, for the main hall and stair doors, were 7 ft. 2½ in. above the concrete floor, and for all other rooms 6 ft. 11 in.

The resulting frame proved to be very irregular and involved numerous different combinations of adjacent slab spans. The same was true of the beam system, since the column locations were restricted to out-of-the-way corners or closets. The restrictions fixed the type of framing to two-way solid concrete slabs, not more than 4½ in. thick, the spans of which were established by the sizes of the rooms. The larger rooms are 13 by 18½ ft., and the spans vary from that upper limit down to 4 ft.

Most of the columns had two of their faces fixed by partitions with which they had to be aligned. As a result, their axes from floor to floor, where their sizes had to be increased, did not register over each other. In general, an attempt was made to keep a 12-in. dimension for one side of the columns, the other side being lengthened as required to secure the necessary section. Although most columns are square or rectangular, some L-shaped columns were required where special clearance had to be met.

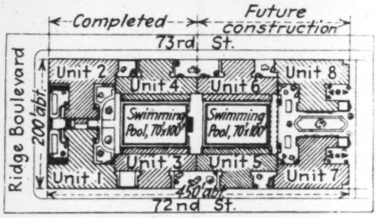


Fig. 2—Ground plan of Flagg Court Apartments, showing final development and the four units completed.

So far as construction was concerned, the restrictions made it mandatory to design a frame that would be as free from cracks as possible. Also because the concrete surfaces formed the finished ceilings the lines and levels had to be maintained with a greater degree of precision than is usually the case. Forms had to be made tight and smooth enough to produce the necessary surface texture.

In designing the concrete frame the moments were computed on the basis of the elastic theory, and the shears and column reactions were based on the moments thus determined. The slabs were proportioned in accordance with the method that is incorporated in the proposed New York City building code, which enables two-way slabs to be designed for any span relation of adjacent slabs or unequal loading, and is based on fundamental principles rather than arbitrary coefficients.

The columns were investigated for bending due to unequal adjacent spans or eccentric loading. The extent of the tensile steel in all members was determined by the negative-moment requirements for each individual case. As a result, in most beams the steel extended considerably beyond the fifth point.

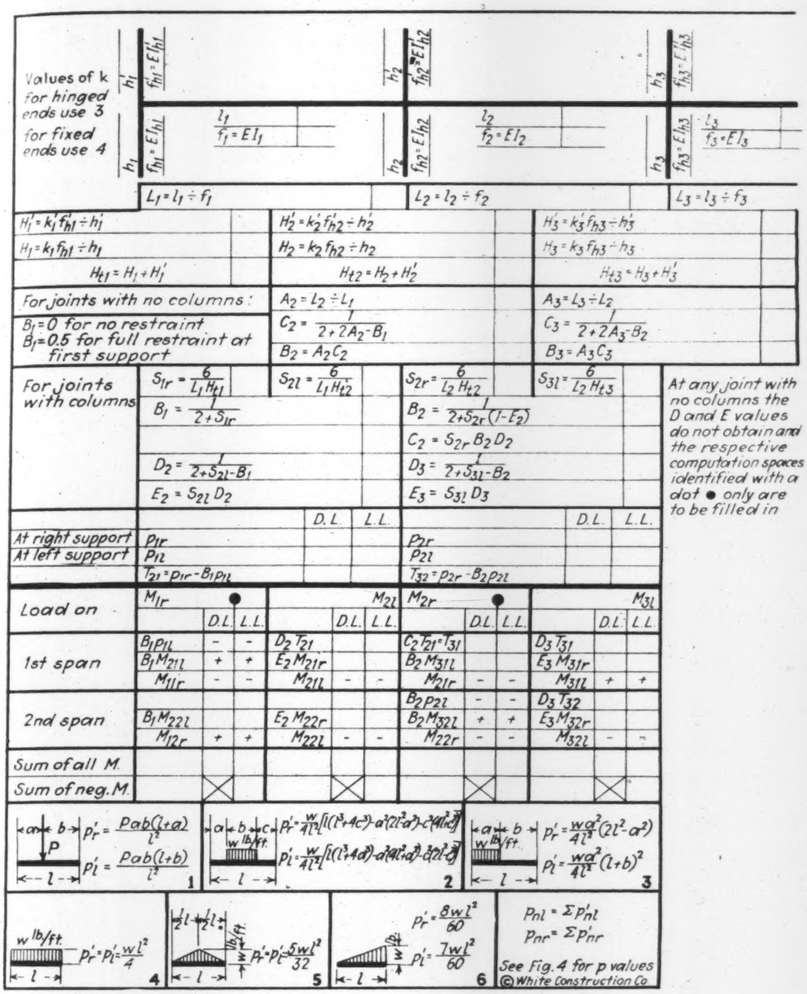

Values of k for hinged ends use 3 for fixed ends use 4
$f_{h1}=EI_{h1}$
$f_{h2}=EI_{h2}$
$f_{h3}=EI_{h3}$
$f_1=EI_1$
$f_2=EI_2$
$f_3=EI_3$
$L_1=l_1\div f_1$
$L_2=l_2\div f_2$
$L_3=l_3\div f_3$
$H_1'=k_1'f_{h1}'\div h_1'$
$H_1=k_1f_{h1}\div h_1$
$H_{t1}=H_1+H_1'$
For joints with no columns:
$B_1=0$ for no restraint
$B_1=0.5$ for full restraint at first support
$A_2=L_2\div L_1$
$C_2=\frac{1}{2+2A_2-B_1}$
$B_2=A_2C_2$
For joints with columns
$S_{1r}=\frac{6}{L_1H_{t1}}$
$B_1=\frac{1}{2+S_{1r}}$
$D_2=\frac{1}{2+S_{21}-B_1}$
$E_2=S_{21}D_2$
At any joint with no columns the D and E values do not obtain and the respective computation spaces identified with a dot ● only are to be filled in
At right support
At left support
$T_{21}=p_{1r}-B_1p_{1l}$
Load on
1st span
2nd span
Sum of all M.
Sum of neg. M.
D.L. L.L.
$p_r'=\frac{Pab(l+a)}{l^2}$
$p_l'=\frac{Pab(l+b)}{l^2}$
$p_r'=p_l'=\frac{wl^2}{4}$
$p_r'=p_l'=\frac{5wl^2}{32}$
$p_r'=\frac{8wl^2}{60}$
$p_l'=\frac{7wl^2}{60}$
$p_{nl}=\Sigma p_{nl}'$
$p_{nr}=\Sigma p_{nr}'$
See Fig. 4 for p values
© White Construction Co.

**Fig. 3—Schematic tabulation of formulas developed by the designers and used in determining the end moments of the horizontal members of the frame.**

A system of individual straight bars at the top and bottom of the beams was used instead of bent bars, the top bars crossing over the tops of the columns. This arrangement eliminated any congestion of bars over the columns and permitted the use of rail-steel reinforcing members, since no bending was required. Straight bars were also used for slab reinforcing. When hooked bars were necessary, as where floors were anchored to wall beams, horizontal loops of intermediate-grade steel were used. The sizes of the columns, because of the small load increments, were changed only at every second story and, because of the low ceiling heights, the column steel reinforcement was assembled in two-story lengths. The bending of the column rods was obviated by using splice dowels at every other story.

## Design procedure

In working out the design calculations an interesting procedure, partly tabular and partly graphical, was used. The aim of this procedure is to reduce to their simplest forms the successive operations in determining the moments in continuous beams and rectangular frames, to eliminate all consideration of signs, to tabulate the steps in a logical sequence, and to make the procedure cover all possible cases arising in building work.

In Fig. 3 the successive operations are arranged in the order in which they are performed to obtain the moments at the supports. Fig. 4 gives convenient graphs for obtaining the load factors $p$ in the equations given in Fig. 3 for unsymmetrical loadings. For symmetrical loadings the load factors are easily computed from the formulas.

The limitations to which the method is restricted are as follows:

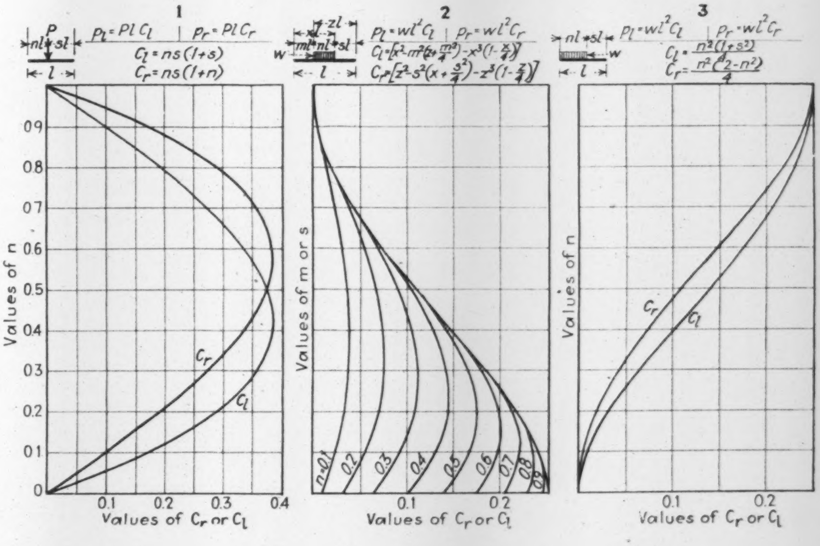
1. The length of girders and columns is measured from center to center of joints.
2. The effect of width of support is not considered.
3. Resulting moments obtain at the center of the supports.
4. Beams are horizontal and columns vertical.
5. Moments of inertia of members are constant along their entire length.
6. The joints suffer no linear displacements.

According to R. L. Bertin, chief engineer of the White Construction Co., who developed the method, its application is divided into three parts:

1. The determination of the constants, based on the relative stiffness and deformations of the various members, namely the *H, L, S, A, B, C, D* and *E* values for the various members of the frame.
2. The determination of the load factors for each beam, namely the *p* and *T* values.
3. The determination of the moments at the supports by combining the stiffness and load factors.

Referring to Fig. 3, the first step is to obtain the *EI* values for each mem-

**Fig. 4—Diagrams used for computing values of load factors *p* for cases 1, 2 and 3 shown at the bottom of Fig. 3.**



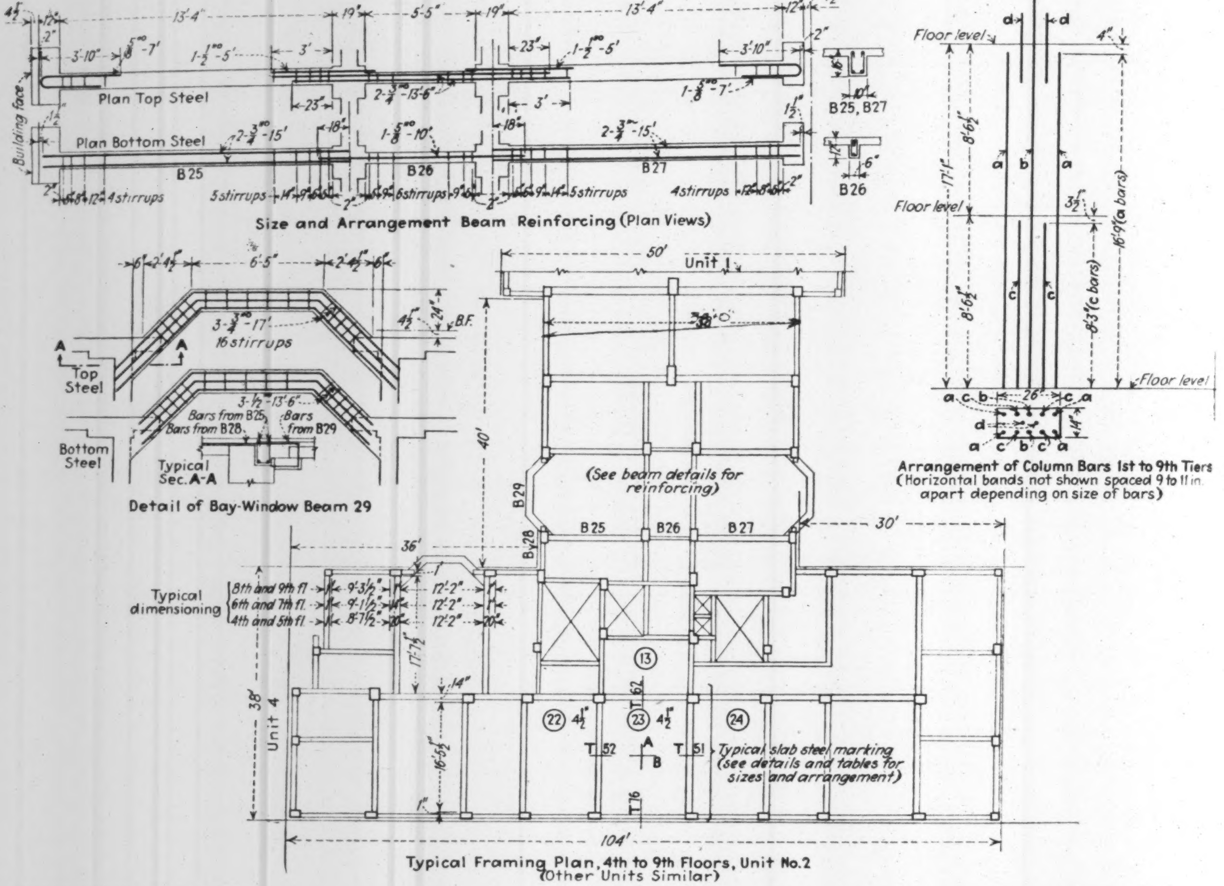
ber, expressing them for convenience in the form of ratios to the value of $EI$ of any one member taken as unity. The $L$ and $H$ values are then obtained.

In computing the $H$ values, an assumed value of $k$ from 3 to 4 is used, depending upon the degree of restraint at the far ends of the columns. In general, a value of 4, which corresponds to a fixed condition, is sufficiently accurate.

Where columns exist the next step is to solve the $S$ values and then the $B$, $C$, $D$ and $E$ values for each span, starting with those for the first span and progressing from span to span to the end of the system. Where there are no columns at the supports, the $H$ and $S$ values do not exist and the $A$, $B$ and $C$ values in the space allotted to joints with no columns are solved instead. The load factors $p$ and $T$ are then computed for each span, using the formulas given at the bottom of Fig. 3 or the graph shown in Fig. 4, together with the proper value of $B$.

Having established all values necessary for the determination of the support moments, as described above, these moments are then computed for each span loaded separately, as follows:

Starting with the load on the first span, solve the values on the first line of the moment schedule, from left to


Plan Top Steel
Plan Bottom Steel
Building face
B25
B26
B27
B25, B27
4 stirrups
5 stirrups
6 stirrups
13'-4"
5'-5"
19"
12"
3'-10"
1-5/8"-7'
1-1/2"-5'
2-3/4"-13'-6"
2-3/4"-15'
1-5/8"-10'
3'
23"
18"
Size and Arrangement Beam Reinforcing (Plan Views)
6'-5"
2'-4½"
3-3/4"-17'
16 stirrups
3-½"-13'-6"
Bars from B25
Bars from B28
Bars from B29
Top Steel
Bottom Steel
Typical Sec. A-A
B.F.
Detail of Bay-Window Beam 29
50'
Unit 1
30'-0"
40'
(See beam details for reinforcing)
B29
B28
B25
B26
B27
30'
36'
Typical dimensioning
8th and 9th fl
6th and 7th fl.
4th and 5th fl
9'-3½"
9'-1½"
8'-7½"
12'-2"
17'-7½"
16'-5½"
14"
38'
Unit 4
13
22
23
24
T 62
T 52
T 51
T 76
A
B
Typical slab steel marking (see details and tables for sizes and arrangement)
104'
Typical Framing Plan, 4th to 9th Floors, Unit No.2 (Other Units Similar)
Floor level
d
a
b
c
8'-6½"
17'-1"
16'-9"(a bars)
8'-3"(c bars)
3½"
4"
26"
14"
Arrangement of Column Bars 1st to 9th Tiers (Horizontal bands not shown spaced 9 to 11 in. apart depending on size of bars)

Schedule of Bottom Steel

| Slab Mark | B Bands | | | | A Bands | | | |
|---|---|---|---|---|---|---|---|---|
| | Spacing | Size | Length | Number | Spacing | Size | Length | Number |
| 23 | 5½" | 3/8" | 12' | 37 | 9" | 3/8" | 17' | 16 |

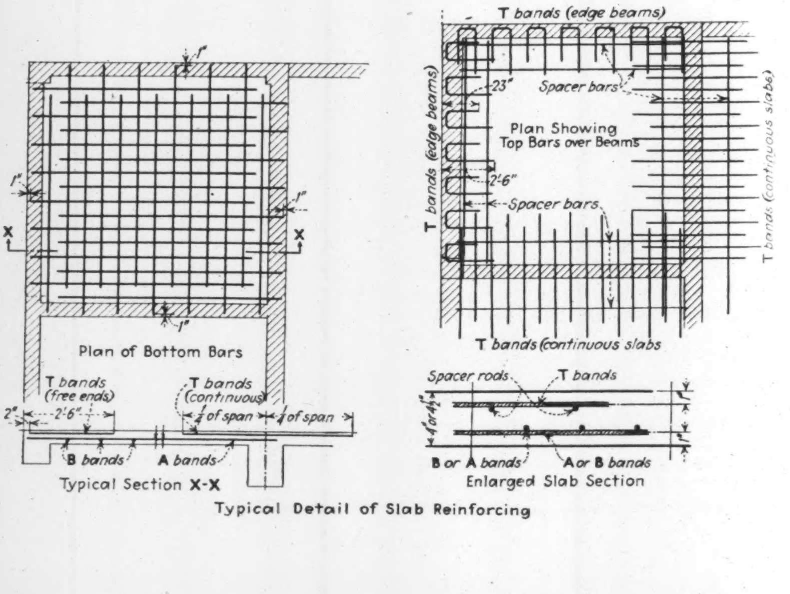
Schedule of Top Steel (Loop bars over edge beams)

| Band Mark | Reinforcement | | | | 3/8 spacer rods |
|---|---|---|---|---|---|
| | Spacing | Size | Length | No. of band rods | |
| T 76 | 10" | 3/8" | 4' | 7 | 2-12' |

Schedule of Top Steel (Interior Bands)

| Band Mark | Reinforcement | | | | Slab No. | CL of beam | | Slab No. | Spacer rods |
|---|---|---|---|---|---|---|---|---|---|
| | Spacing | Size | Length | Number | | | | | |
| T51 | 6" | 3/8" | 5'-6 | 34 | 23 | 3'-3" | 2'-10" | 24 | 2-17' |
| T52 | 7" | 3/8" | 5'-6 | 29 | 22 | 3'-0" | 3'-3" | 23 | 2-17' |
| T62 | 6" | 3/8" | 6' | 24 | 23 | 4'-6" | 2'-3" | 13 | 2-12' |

**Fig. 5—Floor plan and structural details, showing reinforcing-steel sizes and arrangement. Unbent rail steel bars were used except for wall anchors, which are U-bends of intermediate-grade steel.**


Plan of Bottom Bars
X
1"
T bands (free ends)
T bands (continuous)
2"
2'-6"
¼ of span
B bands
A bands
Typical Section X-X
T bands (edge beams)
T bands (continuous slabs)
23"
Spacer bars
Plan Showing Top Bars over Beams
2'-6"
Spacer rods
T bands
4 or 4½"
B or A bands
A or B bands
Enlarged Slab Section

Typical Detail of Slab Reinforcing

right, to the end of the system or as far as the magnitude of the resulting value has any significance. Having obtained the value for the last span, compute the values on the second line, from right to left, adding the value thus found to the ones directly above them. The negative moments at the right and left of the center of support are represented by the sum of these values, except in the case of the moments to the right of the support on the left side of the loaded spans, where these values are to be deducted instead of added. This operation of moment determination is repeated with the next span loaded and so on to the end of the system. When all support moments for each individual span have been computed, ordinary procedure is used in obtaining the maximum moments and shears for all parts of the system. The tabulation in Fig. 3 may be extended to any number of spans by extending the equations to the right and changing the sub-values to correspond with the span number identified from the first span at the left.

### Method of construction

Five-ply Douglas fir plywood was selected for the under side of the slab forms in building the Flagg Court Apartments. This material is manufactured in panels 4 by 8 ft., the plys being glued together with a waterproof glue, and the surfaces given an oil treatment to render the plywood resistant to moisture. No satisfactory method of covering the joints was developed, and the small fin that remained on the ceiling had to be chipped and rubbed down. Ordinary ⅞-in. tongue-and-groove lumber was used for the sides of columns and beams. A few of the beams and column sides not covered by partitions had to be rubbed down where the imprint of the wood grain on the concrete was too pronounced.

The ten tiers in the building were built with form panels for only one complete floor plus about the equivalent of half a floor for replacements. Re-use caused the edges of some of the panels to wear, and these edges had to be cut back an inch or so and filler strips of new plywood added.

The plywood panels were ½ in. in thickness and sufficiently strong to span 15 in. without deflecting unduly under the weight of 5 in. of concrete, so that the panels not only served as a lining material but also as the structural part of the form.

The design of the structural concrete was based on 2,500 lb. per sq.in. ultimate strength for the floors and 3,000 lb. for the columns. In selecting the brand of cement for the work the early strength characteristic was one of the chief deciding factors. Preliminary tests were made to establish the proportions of the concrete, and the mixtures finally adopted were such as to produce strengths 80 per cent of the ultimate in seven days, in order to reduce the period between pouring and stripping to a safe maximum. Automatic measuring devices, consisting of a volumetric batcher for the coarse aggregate and an inundator for the sand, were used at the mixer. Yield factors were checked every day, and 7- and 28-day cylinders were made and tested for each floor.

In constructing the building special care was exercised in locating construction joints. The frequency and location of such joints was determined with a view to keeping the shrinkage stresses below the point where cracks might be expected. The use of columns with low rigidity at the corners was found to be desirable in reducing corner cracks.

Comparative estimates made by the designers convinced them that when the savings effected by the reduced total height of walls, due to the thinness of construction between floors and ceilings, and by the elimination of ceiling plaster, floor fill and finish are taken into account, the cost of the entire structure is below that of any other fireproof construction. Cost analysis of reinforced-concrete skeletons of similar design for a variety of plans seemed to indicate small differences for widely varying layouts, provided that the largest room sizes did not vary materially from one another and that the magnitudes of the various structures were about equal.

Brick walls were used, and in the opinion of the designers are to be preferred to concrete walls on the score that the expansion and contraction differential between concrete walls and floors induces stresses in both that are not desirable and tends to produce cracks. Also the heat lost through concrete walls probably would be considerably greater than through the 4-in. brick backed up with the 8-in. hollow terra cotta tile that was used.

Ernest Flagg, the owner of the building, was also architect and builder. White Construction Co. had the subcontract for the reinforced-concrete construction, which included also the design of the structural frame.

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# Topographic Surveying as an Aid to Unemployment

By **C. H. Birdseye**
*Engineering Assistant to the Director, U. S. Geological Survey, Washington, D. C.*

UNEMPLOYMENT among engineers is still widespread, so that consideration of any program that will put unemployed engineers to work on worth-while projects that must be undertaken in the near future is amply warranted. Most of these men are specialists and they naturally desire reinstatement on jobs for which they are specially trained. They probably fit in best on public-works projects, as has been advocated by *Engineering News-Record* to the extent of immediate need and available financing. But there are not enough projects of this nature for which funds are now available to absorb all of the engineers who are still out of work.

There are, however, many areas for which topographic and geodetic surveys are needed before proper utilization of the land or proper planning of public works can be accomplished, and these two activities should not be postponed even in this time of need of reduction in public expenditures. Every dollar wisely spent now on this sort of work will save many dollars in the near future. It is apparent therefore that badly needed topographic and geodetic surveys offer a means of putting at least a few unemployed civil engineers to work without a large expenditure of funds, and to pay them enough to feed, house and clothe them and their families. These are the essential needs, and of course meeting them by employment rather than by public or private doles is better for all concerned in the matter.

### Surveys in New York

Last winter the Professional Engineers' Committee on Unemployment in New York City conceived the idea of cooperation with the Geological Survey in extending topographic and geodetic surveys over the metropolitan area around New York and of employing on this work engineers who had been out of work so long that they and their families were faced with suffering from lack of food and shelter.

The Geological Survey agreed to this proposal and selected a project comprising a series of 7½-min. quadrangles on both sides of the Hudson River just north of the city limits. The first operations involved the geodetic control of these quadrangles by lines of transit traverse and spirit leveling, and 30 unemployed engineers were started on the project early in January, 1932, most of them being kept on the job until late in June. Three transit-traverse parties of six men each and two spirit-level parties of four men each were formed. Four men were assigned to computing the traverse data and adjusting the level lines, office space being assigned without charge by one of the members of the committee.

The work was new to most of the engineers as both geodetic traversing and leveling differ considerably from ordinary transit and level operations. The project was not considered one of training for these men but one designed to spread employment and secure as much productive work as possible. The men, therefore, were not rotated in the different positions in the parties, but the most adaptable men were selected for the jobs of instrument men, recorders and computers, and the less adaptable were assigned to jobs as rodmen, chainmen and linemen, all being paid the same wage of $4 per day. The men were quartered at their homes in New York City and taken each day from mobilization points at the ends of 5c. fare zones to and from the points of field work in automobiles.

Six engineers of the Geological Survey were detailed at the start to instruct the new men in the operations of horizontal and vertical control surveys and to supervise the work of the field parties and office computations; but three were soon withdrawn, leaving only one to supervise the work of the transit parties, one the work of the level parties and one that of office computations. In this work 470 miles of traverse and 340 miles of leveling were executed, and practically all of the computations and adjustments were made in the field.

Toward the close of the work four topographic mapping parties were organized, to each of which was assigned one Geological Survey engineer as topographer and two unemployed engineers as assistants. In all, 43 square miles were mapped on a scale of 1:24,000, with a contour interval of 10 ft.

The Professional Engineers' Committee on Unemployment, through the state public works department, obtained $15,000, and the Geological Survey allocated an equal amount, so that a total of $30,000 was available. Of this $12,900 was spent on geodetic traverse, $7,840 on spirit leveling, $3,830 on computations and $5,620 on topographic mapping. Moreover $26,370 was actually expended in the field and only $3,820 in the Washington office. A further analysis of the field expenditures shows that only $5,170 was spent on topographic mapping, while $21,200 was spent on traverse, leveling and computations, of which $13,480, or about 64 per cent, was paid in salaries to local unemployed engineers. In all, $14,974, or $4 more than the entire state expenditure, was paid as salaries to the unemployed engineers. In addition, $5,035, or 19 per cent of the total field expenditure, was spent locally for subsistence and supplies, all of which went at least indirectly to relieve unemployment.

It is apparent, therefore, that the project was a decided factor in the relief of unemployment, and as such was well worth while. From the point of view of results obtained the picture is not as favorable, but cheap work or rapid progress was not anticipated because much of the work had to be done in the winter with short days and much bad weather. The average cost of the traverse work was $22 per linear mile, which is about 35 per cent more than the average cost of similar work by trained engineers of the Geological Survey. Likewise, the average cost of the leveling was $18 per linear mile, which is about 50 per cent more than the average cost of similar work. Moreover, the average cost of the topographic mapping was $123 per square mile, which is at least 25 per cent over average costs of similar work.

### Mapping Pennsylvania coal regions

Pennsylvania became interested in the project and allotted $6,000 for cooperative control surveys of ten 7½-min. quadrangles in the anthracite coal region of that state. Later the state reduced this allotment to $3,500, which was met by an equal allotment by the Survey. In all, $6,980 was expended, of which $3,558, or 51 per cent, was paid in salaries to local unemployed engineers. In addition, about $1,200 was expended locally for subsistence and supplies, so that 68 per cent of the total cost of the work was expended locally, the remaining 32 per cent going for salaries of the federal engineers, much of which was expended locally. The state should be gratified to know that every dollar it spent went to the pay of unemployed men, and that for every dollar it spent it received $2 worth of needed work which it would have to accomplish soon.

### Survey planned at St. Louis

During this year St. Louis is planning to follow much the same procedure, except that in this case the city is interested in large-scale mapping of the suburban areas of the city and plans to execute the topographic mapping of the area by utilizing its own unemployed personnel in such a way that the original sheets will be a part of the city plan map. The part played by the Geological Survey will be in supervising the geodetic traverses, leveling and office computations, in contouring the sheets, in correcting the details of culture by aerial photography and in assembling the data in final form in a series of standard sheets on a scale of 1:24,000. In doing this the city will have a series of sheets on a scale of 1:2,400 for detailed planning and, with the Geological Survey, will have a series of smaller-scale standard sheets for general planning on a scale of 1:24,000, both at a combined expenditure of what it would cost to obtain the standard sheets.

Another city was interested in a similar plan and proposed a cooperative scheme involving surveys on an extremely large scale for straightening the channel of a stream and improvement of the stream valley. This project was so far beyond the authorized scope of the topographic mapping activities of the federal government that the Survey was forced to decline to participate because it would have been a straight federal contribution to work that would not constitute a part of its authorized mapping program.

Most of the states and many of the large metropolitan districts could undertake work similar to that outlined herein as a means of unemployment relief. Such work produces much-needed map information and will, as indicated herein, result in an unusually large percentage of the money being spent directly for labor and local subsistence—the main objective in the use of relief funds.

## Italy Subsidizes Land Reclamation

AN EXTENSIVE land-reclamation program is being carried out by the Italian government, to reclaim by irrigation and drainage more than 2,000,000 acres of now sparsely populated land. The ultimate cost of the work is estimated at $400,000,000. The purpose of the program is many-sided, including increase of rural population and removal of excessive city population, placing the nation on a self-contained agricultural basis and creating a background for sounder industrial development.

Subsidy of the settlers is an essential feature of the plan. Irrigation and drainage improvement work are supported by the government up to seven-eighths the total cost, while the owner of the land directly benefited may pay the remaining one-eighth in as long a period as 30 years. Under this plan, however, the owner is obligated to make beneficial use of the land improved. Boards of appraisal in the various districts determine the value of the land of delinquent owners, based on unimproved prices, and the government then buys such lands and resells them to individuals prepared to improve them.

For the reclamation either of arid or of swamp land or the improvement of partly reclaimed land the usual method is to form a corporation organized somewhat like an irrigation district. This district either works out its own plan or requests the government to develop a plan of improvement. Financing varies with the individual case. In some instances the government carries the entire cost of the major work, including canal, road and bridge construction, while the district finances the secondary work, borrowing the money from the government on long-term refunding. The work now under construction runs to a cost as high as $200 per acre.

The ultimate aim of the land-development planning of the kingdom is to bring under intensive cultivation every acre of arable land in the country. Through the work so far carried out a large decrease in unemployment has been brought about, and this effect is expected to continue.

# A 320-Ft. Concrete Arch on Scenic Route Along California Coast

**Bixby Creek Bridge in difficult location on exposed coast necessitates timber falsework 240 ft. high—Selection of site, arch design and construction operations**

By Stewart Mitchell
*Construction Engineer of Bridges, California Division of Highways*

A DEEP GORGE and a location on an exposed coast imposed special and difficult problems in the construction of the 320-ft. Bixby Creek concrete arch by the California division of highways about 15 miles south of the Monterey Peninsula. Timber falsework 240 ft. high was required, and wind and wave action had to be resisted by the falsework foundations. The arch—the longest on the California highway system—is of open-spandrel design with ribs 4½ft. wide and 5 ft. and 8 ft. 9 in. thick at the crown and spring line, respectively. Nine 40-ft. girder approach spans complete the total 714-ft. length of structure. Scenic requirements for a new coast route and the avoidance of tortuous alignment, of adverse grades and of added distance justified the crossing of several deep ravines with concrete arches, of which the 320-ft. Bixby Creek span is the longest.

The present highway, known as the Coast Route between San Francisco and Los Angeles, passes through Monterey County on an inland location along the Salinas River Valley separated from the ocean by the high Santa Lucia Range. This route in general follows the trail of the early Spanish padres, "El Camino Real," and presents much of historic interest. The motoring public, however, has for many years expressed a desire for an alternate route along the ocean shore, which would provide relief against the summer heat of the interior valley and a change in scenery. Such a highway is now under construction, necessitating a location along the rugged Monterey County coast where the mountains rise precipitously from the ocean and are cut at frequent intervals by deep canyons. When completed, the new location will overlook the ocean practically the entire distance between San Luis Obispo and the Monterey Peninsula, affording a magnificent scenic highway for about 150 miles.

To obtain the most advantageous alignment and grades, the new highway is located along the steep slopes facing the ocean, running for much of the distance at a considerable elevation above the beach to avoid the sheer wave-formed cliffs and to take advantage of occasional bench lands. This route necessitates the crossing of the numerous intersecting ravines with high structures or deep fills over culverts. Four reinforced-concrete arch structures have recently been completed along the newly graded section at the northerly end of this route, each consisting of a main arch span with necessary girder-span approaches. All structures have a roadway width of 24 ft., which is the minimum standard for state highways. The theoretical span and maximum height above streambed are:

| | Span | Height |
|---|---|---|
| Granite Canyon | 127 ft. | 105 ft. |
| Garrapata Creek | 150 ft. | 95 ft. |
| Rocky Creek | 225 ft. | 150 ft. |
| Bixby Creek | 320 ft. | 265 ft. |

The Bixby Creek Bridge is particularly notable because of the length of span, height above streambed and the construction problems caused by weather and local conditions.

### Location

The site is a V-shaped canyon about 100 ft. wide at the bottom, with side slopes averaging 1:1 to 1½:1 (Figs. 1 and 2). The slopes consist mainly of altered and disintegrated granite of varying degrees of hardness, with superimposed limestone and shales, all of which are faulted and seamed. The location is so close to the beach that falsework or other supports in the bottom of the canyon are exposed to wave action during storms. The site is also directly exposed to gales from the ocean, which blow with terrific force during the winter. In the summer there are long periods of fog which, together with the salt spray, make the painting and maintenance of a steel bridge unsatisfactory and expensive.

Owing to the expensive construction that evidently would be necessary at the Bixby Creek site and for a half mile north (this section also includes the bridge over Rocky Creek), other possible preliminary locations were run out and studied. The most promising alternative line involved turning the highway up Bixby Creek Canyon a few hundred feet to a shorter bridge crossing site, then, by means of a 900-ft. tunnel, cutting through the ridge be-

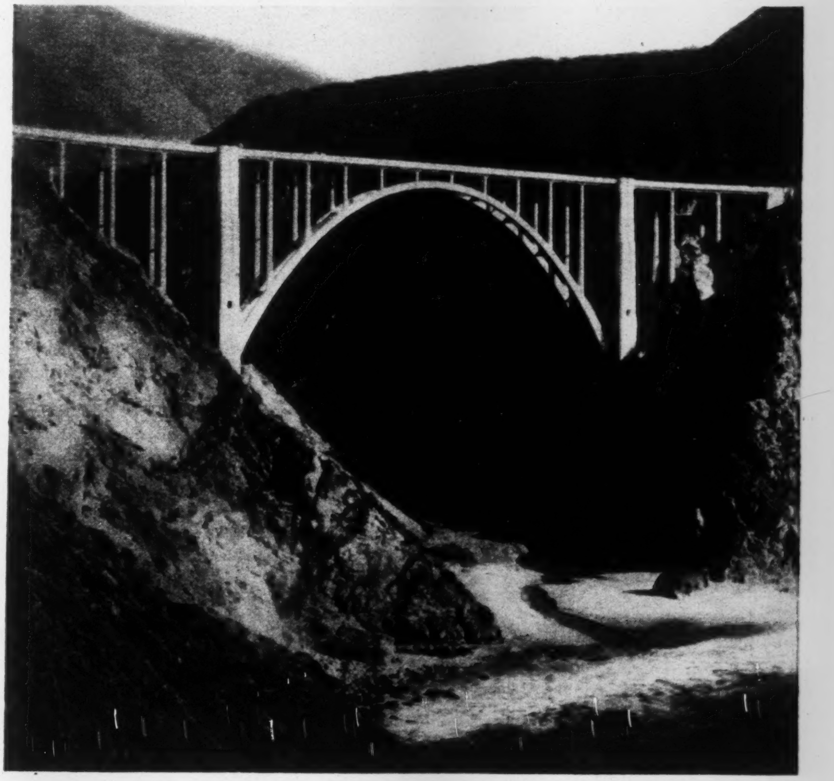


Fig. 1—Scenic setting for California's longest concrete highway arch on a new oceanfront route. The location saved tortuous alignment, grades and distance.

tween Bixby and Rocky Creek and crossing the latter by a culvert under a high fill. The reasons for choosing the present line with its more expensive bridge construction and somewhat higher estimated total cost were: (1) better alignment, (2) more scenic location, (3) avoidance of a long dark tunnel with no convenient means of providing lighting, and (4) uncertainty in the cost of constructing and maintaining a tunnel through badly broken and faulted formation.

After the selection of the site came the choice of the most economical and suitable structure, with consideration of a concrete arch and some form of long-span steel structure of either arch or cantilever type. A steel structure built out from each bank as a cantilever would have an advantage over a concrete arch erected upon falsework because of freedom from danger of being wrecked by the waves during a storm, but the hazard from the effect of high winds would probably be as great. It is probable that a steel structure could have been built at a slightly lower first cost than the concrete-arch type, but the annual cost during its expected life would have been considerably greater owing to the high cost of keeping the steel painted.

A third plan was to design a structure having steel-arch ribs that could be erected without the use of falsework and later be incased in concrete, using forms supported from the steel rib itself. Such a structure was actually designed and bids were secured in competition with a reinforced-concrete arch erected upon falsework in the usual manner, but the latter structure was found to be cheaper. The low bid for the steel-arch rib design was 8 per cent higher than the low bid for the falsework type of construction shown in the accompanying table.

A complete preliminary investigation of the foundations was not made, since a thorough prospecting of all the footings would cost about as much as their complete excavation. Sufficient information, however, was obtained to determine the probable average depth of footings and to assure the dimensions of the arch span. Actual excavation for the footings averaged about the same as estimated, but it was found advisable to do extra work on flattening and removing the slopes above the arch abutments, since the material showed cracks and signs of slipping after the original formation had been disturbed by the excavation.

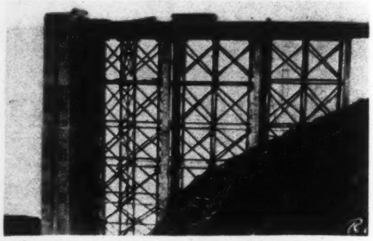


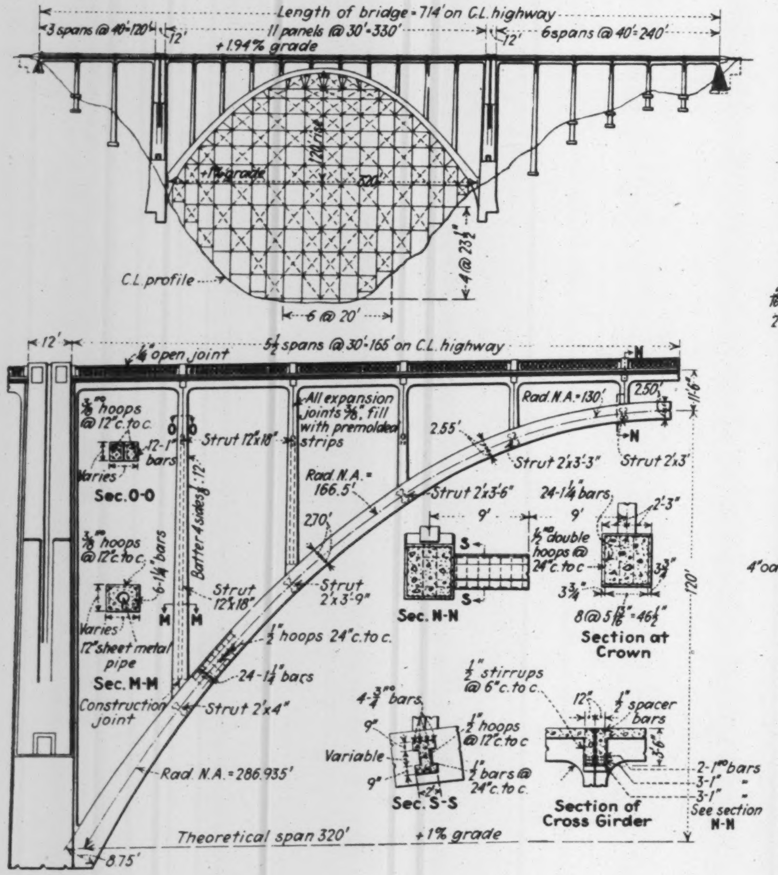
Fig. 3—Approach girder spans also required high falsework and long columns. All spans are 40 ft., and there are three at one end and six at the other end of the arch.

## Final design

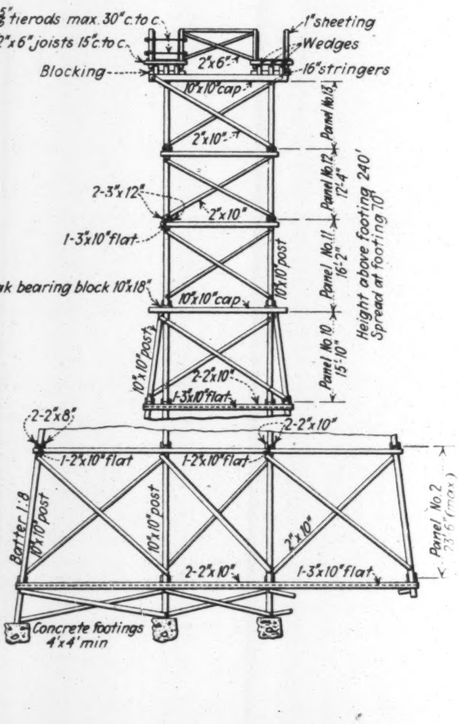
The design provides an open spandrel arch of 330-ft. span between pilasters at the roadway level, with three 40-ft. girder approach spans at the south and six similar spans at the north. The pilasters are 12 ft. wide at roadway level, making a total length of 714 ft. measured along the center line of the bridge. Both approaches are on curves of 500-ft. radius, and the intervening tangent extends over most of the central arch span.

The 320-ft. arch has a rise ratio of 0.375 and consists of two ribs 5 ft. thick at the crown and 8 ft. 9 in. thick at the spring line. Each rib is 4 ft. 6 in. wide, and cross-struts are provided at each column point. The amount of reinforcement is 1.15 per cent at the crown, and the two ribs contain 858.6 cu.yd. of concrete, including struts and column bases. The superstructure consists of standard 30-ft. girder spans supported on columns that are split for

Fig. 2—Design features of arch and falsework for 320-ft. Bixby Creek Bridge on new Coast highway in California.



Length of bridge = 714' on C.L. highway
3 spans @ 40' = 120'
12'
11 panels @ 30' = 330'
12'
6 spans @ 40' = 240'
+1.94% grade
+1% grade
120'
320'
4 @ 23½'
6 @ 20'
C.L. profile
12'
¼" open joint
5½ spans @ 30' = 165' on C.L. highway
N
⅜" hoops @ 12" c. to c.
12-1" bars
Varies
Sec. O-O
O
Strut 12"x18"
All expansion joints ⅜" fill with premolded strips
Rad. N.A. = 130'
2.50'
11'-6"
2.55'
Strut 2'x3'-3"
Strut 2'x3'
Rad. N.A. = 166.5'
Strut 2'x3'-6"
24-1¼" bars
2'-3"
2.70'
9'
9'
½" double hoops @ 24" c. to c.
3¾"
3¾"
8 @ 5 1/16" = 46½"
Section at Crown
S
Sec. N-N
⅜" hoops @ 12" c. to c.
6-1¼" bars
Batter 4 sides 1:12
Strut 12"x18"
Strut 2'x3'-9"
M
Varies
12" sheet metal pipe
Sec. M-M
½" hoops 24" c. to c.
24-1¼" bars
Construction joint
Strut 2'x4'
½" stirrups @ 6" c. to c.
4-¾" bars
9"
Variable
9"
½" hoops @ 12" c. to c.
½" bars @ 24" c. to c.
Sec. S-S
12"
½" spacer bars
2-1" bars
3-1" "
3-1" "
See section N-N
Section of Cross Girder
Rad. N.A. = 286.935'
Theoretical span 320'
+1% grade
8.75'
120'



⅝" tierods max. 30" c. to c.
2"x6" joists 15" c. to c.
Blocking
1" sheeting
Wedges
16" stringers
2"x6"
10"x10" cap
2"x10"
Panel No. 13
Panel No. 12 12'-4"
2-3"x12"
2"x10"
1-3"x10" flat
Panel No. 11 16'-2"
10"x10" post
Height above footing 240'
Spread at footing 70'
4" oak bearing block 10"x18"
10"x10" cap
Panel No. 10 15'-10"
10"x10" posts
2-2"x10"
1-3"x10" flat
2-2"x8"
2-2"x10"
1-2"x10" flat
1-2"x10" flat
Batter 1:8
10"x10" post
10"x10" post
2"x10"
2-2"x10"
1-3"x10" flat
Panel No. 2 23'-6" (max.)
Concrete footings 4'x4' min

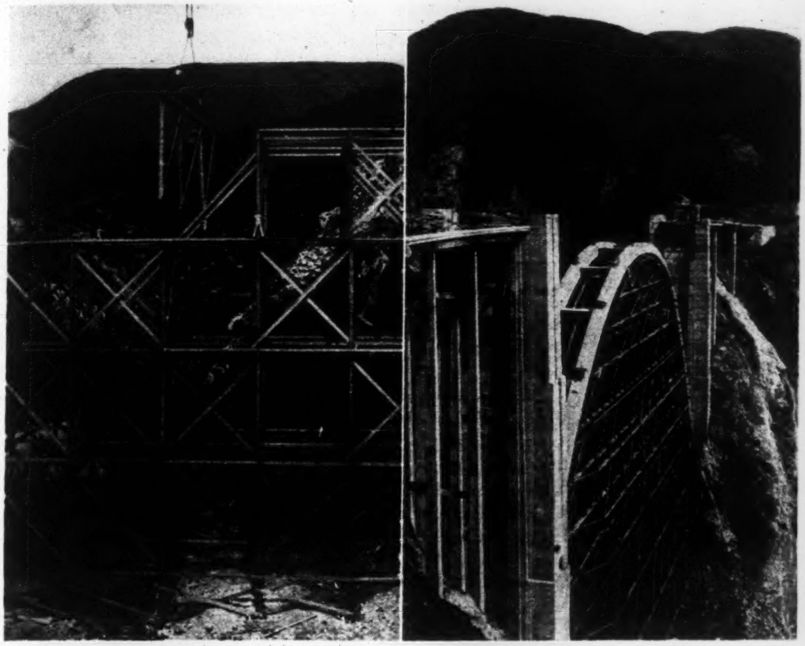


**Fig. 4—Spectacular trestle-type timber falsework was erected in two months. Bents were framed and set by cableway (left) at a rate of twelve per day. Maximum spread of the batter posts was 70 ft. (right), and no guys were used.**

part of their length near the top to take care of deck expansion.

Concrete in the deck, including curb, is about 1.5 cu.yd. per linear foot of span, and the columns add about 0.5 cu.yd. per linear foot. The arch abutments contain 2,216 cu.yd. of concrete, and the hollow piers that rest on them and provide architectural treatment and lateral stiffness at the highest point of the superstructure contain 1,894 cu.yd. The appraches are standard 40-ft. reinforced-concrete girder spans and have no unusual features except possibly the high columns and falsework required near the arch span (Fig. 3).

The maximum footing pressure under the arch abutments is 11 tons per sq.ft., which is conservative for the type of supporting rock. Maximum pressure on the column footings resting on rock or compacted gravel is only 4.5 tons per sq.ft.

Contract was awarded to the low bidder on Aug. 13, 1931, and concrete placing started Nov. 4. Because of the lateness in the season the contractor chose to postpone erection of the arch centering until spring after most of the danger from high winds had passed. The pouring of the key sections of the arch ribs was completed on June 26, 1932, and the structure was dedicated on Nov. 27.

The nearest railroad point was at Monterey, 21 miles away, and until the structure was nearly completed all lumber and cement was trucked over a one-way road with numerous hairpin turns. Aggregate was obtained from a screening plant set up on the Big Sur River about 8 miles south of the site. The framing yard and camp were located at the north side of the canyon, and the concrete plant was set up on the other side, nearer the aggregate supply. Handling of forms, falsework material and concrete material was done by cableway.

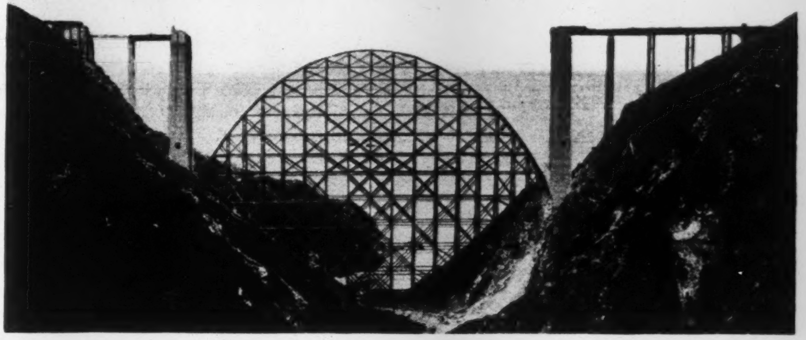


**Fig. 5—Ready for arch concrete. The falsework was 240 ft. high above the stream, and the rib concrete caused 5/16-in. settlement due to crushing at the joints and 1½ in. from elastic deformation.**

Aggregate trucks discharged directly into bunkers, which fed batchers (subsequently equipped with weighing devices) emptying through steel hopper directly into a 1-yd. mixer. The mixer discharged into bottom-dump buckets that were transported by the cableway to hoppers on the forms, from which short chutes led to the place of deposit.

A fixed quantity of cement per cubic yard for each class of concrete was specified (see table). The raw aggregate material contained many large-size boulders that were crushed to conform to the specified maximum. This added about 50 per cent of crushed material to the coarse aggregates. Sand was of good quality but was somewhat coarse in grading and no cheap source of fine sand was available. As a result rather harsh working concrete was obtained at times but was later remedied to a great extent by the use of plastic, early-hardening cement at no increase in cost. This cement is finely ground and has a specific gravity about 0.95 that of standard cement. The greater bulk furnished for the same weight as standard cement, and the better grading of its particles accounted for the increased plasticity. Another important advantage in the use of this cement with the available aggregate was the absence of free water. The water-cement ratio was kept below 0.80, the mix being made as dry as possible consistent with the conditions under which it had to be placed.

FINAL QUANTITIES AND COST FIGURES

| Item | Quantity, Cu. Yd. | Bid Price | Cost |
|---|---|---|---|
| Structure excavation. | 4724.56 | $2.60 | $12,283.86 |
| Class A (6 sacks/yd.) (Arch ribs) | 856.36 | 39.00 | 33,398.04 |
| Class A............ (Excluding arch ribs) | 3069.58 | 20.10 | 61,698.56 |
| Class F (7 sacks/yd.) (colums) | 428.07 | 22.24 | 9,520.28 |
| Class E (9 sacks/yd.) (Handrail) | 54.69 | 54.00 | 2,953.26 |
| Rubble masonry.... (Retaining walls) | 732.77 | 9.00 | 6,594.93 |
| Miscellaneous items. | .......... | ..... | 1,225.00 |
| Class B (5 sacks/yd.) (Arch abutments) | 2216.41 | 19.00 | 42,111.79 |
| Reinforcing steel, lb.. | 619,912.00 | .047 | 26,656.22 |
| Subtotal contract items............. | | | $196,441.94 |
| Extra work-stripping slopes......... | | | 3,419.32 |
| Total cost....................... | | | $199,861.2 |

### Falsework

The outstanding feature in the construction of the Bixby Creek arch was the use of trestle-type centering for such a high span. According to estimates made by the contractor, comparing timber centering with the use of steel-truss sections set by a high line and fitted together as an arch supporting the forms for the concrete ribs, the steel centering would have to be used for about four similar spans before it would be as economical as the timber.

Footings for the falsework posts under the approach spans were of concrete averaging about 2 ft. square and 18 in. thick. They were poured against solid rock or a gravelly hardpan. The load per square foot on the footings did not exceed 3.5 tons. The falsework timber for the approach spans was Douglas fir of good quality shipped direct from the Northwest. Posts were 10x10-in. timbers spaced 17 ft. c. to c.; 4x16-in. stringers and 10x10-in. caps were used. The bracing was of 2x10-in. plank securely bolted to the posts by ⅝-in. bolts. Use of this large-size material enabled the contractor to re-use it in the arch centering. The deflection allowance in the 40-ft. girder spans was 1¼ in. Camber remaining in the girders was about ¾ in.

The arch centering for this bridge is probably the highest timber falsework used in bridge construction in the state. The height of 240 ft. above footings and the load of the concrete made the foundation problem one of major importance. Suitable rock for the footings was available on the slopes. Tests indicated rock and boulders in the creek bottom, and the contractor's decision was to excavate open holes to suitable material. Pile support was considered, but the contractor decided against this method because of the possibility of striking boulders.

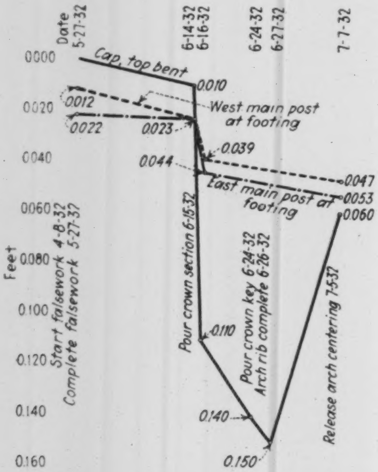


Fig. 6—Record of settlement at highest bent of falsework.

Most of the footings were taken down to gravel and boulders for a maximum depth of 8 ft. A few were stopped short of this stratum on account of the depth, but the material was solid and considered to have sufficient lateral restraint to support the load. The thickness of the footings varied, and footings that could not be pumped dry were tremied. Those on rock were about 18 in. thick, and those on the gravel and boulders in the creek bottom were 2½ ft. or more. The load on the footings in the creek bottom was less than 1.25 tons per sq.ft.

At the maximum section, 240 ft. high, the falsework bent consisted of two 10x10-in. vertical posts 18 ft. apart and two 10x10-in. batter posts, with a total base spread of 70 ft. The batter posts were carried to within three stories of the top, and the remaining section (50 ft. maximum) consisted of the two-post bent. Construction features of the highest bent are shown in Fig. 2. No guy lines were used, and the centering was remarkably free from sway.

The bents for each story were framed on the top of the story just completed. They were picked up by the high line and set into place (Fig. 4). An average of twelve bents was set in a day when the supply of timber was sufficient. Two months was required to put up the falsework. A novel feature of the centering was the construction of the horizontal struts that were built up as H-sections with the webs butting the 10x10-in. posts and the flanges butting the corresponding strut members on either side of the post. Above the springline 3x12-in. planks formed the flanges and a 3x10-in. was used as the web of the section. Smaller-size material was used below the springline and in the make-up of the traverse struts.

All the posts were butt-fitted, except where the batter posts ended. At this level the cap was carried over all four posts and an oak bearing block was placed on top of the cap to increase the bearing area. Particular care was exercised in fitting these posts. No shimming was necessary at any joint. The maximum unit bearing pressure perpendicular to the grain was 470 lb. per sq.in. which occurred only under the upper cap of the two post bent. A good grade of Douglas fir lumber was used, and the average settlement was roughly 1/16 in. per joint.

A series of observations was taken on the cap of the highest bent to note the settlement as the pouring of the ribs progressed. To determine the amount of settlement in the footings, brass tacks were set in all the plumb posts in the creek bottom as soon as the first story was up. Readings were also taken on these during erection of the falsework and when pouring was in progress. These settlement data are shown in Fig. 6. It will be noted that the east footing of the bent settled ¼ in. by the time the falsework was completed. Maximum settlement of this footing was ⅝ in. Readings on the cap taken immediately before and after the crown pour showed that the bent settled 1 in. and that an additional ¼ in. of settlement occurred during the night. The maximum settlement below the cap was 1 13/16 in., of which ⅜ in. was due to footing settlement. When the wedges were released readings showed that the cap had risen 1⅛ in., which indicated that 1/16 in. was due to crushing of the joints and 1⅛ in. was due to elastic deformation.

Observation points were placed on the concrete as soon as the arch ribs were poured and readings were taken during erection of the superstructure forms, pouring of the columns and pouring of the deck system. A change of elevation of ¼ in. was noted during the day due to variations in temperature. The final reading was taken 75 days after the initial one with full dead load of the arch and deck system acting. This final reading on the ribs showed that a ½-in. settlement had occurred caused by cooling shrinkage in the concrete and rib shortening under load.

The Bixby Creek Bridge was built by the California division of highways under the general direction of C. H. Purcell, state highway engineer. F. W. Panhorst, acting bridge engineer, was in charge of the work, with the writer as supervising construction and I. O. Jahlstrom as resident engineer. The contract was carried out by the Ward Engineering Co., San Francisco, with E. C. Panton general superintendent.

* * *

## Prevention of Dry Rot in Buildings

It is much easier to prevent decay in buildings than it is to eliminate the decay due to dry rot after it is once started, according to C. Audrey Richards, pathologist, U. S. Division of Forest Pathology. There is only one general rule to remember: control the moisture content of the wood or, if the conditions of use are such that the moisture content cannot be controlled, use wood treated with a suitable preservative.

In the construction of new buildings the following precautions have been outlined by Mr. Richards in order that decay of buildings may be prevented: (1) build on a well-drained site; (2) secure well-seasoned lumber from a yard where rot in foundations and lumber piles is not tolerated, rejecting any material that is suspected of containing incipient decay; (3) do not allow the selected material to lie on the ground after it has been delivered on the job; (4) untreated lumber should not be allowed to come in contact with the soil or with foundations or walls that are liable to be damp, and should not be embedded in concrete or masonry without leaving ventilation around the ends of the timbers; (5) wood flooring should never be laid directly on the soil or on concrete unless it has been chemically preserved; (6) ample ventilation should be provided so that free circulation of air around the wood will keep the wood dry.

These precautions were given by Mr. Richards in a paper before the recent meeting of the American Wood Preservers Association. The U. S. Division of Forest Pathology is maintained in cooperation with the Forest Products Laboratory at Madison, Wis.

# Pavement Base Strengthened to Keep Horizontal Plane

### Concrete base on unstabilized subgrade designed to preserve level and elevation of bituminous surfacing in case of possible cracking or joint movement

By **H. F. Clemmer**
*Engineer of Tests, District of Columbia, Washington, D. C.*

THE HIGHWAY DEPARTMENT of the District of Columbia recently constructed a section of pavement of sheet asphalt on concrete base, amounting to about 25,000 sq.yd. The pavement carries a considerable amount of heavy traffic, and it was therefore necessary to adopt a high-type rigid design. The bituminous surface was desired to conform with the rest of the street.

A portion of the roadway was new location consisting of both cut and fill, and the rest was over two old fills, one in rather unstable condition.

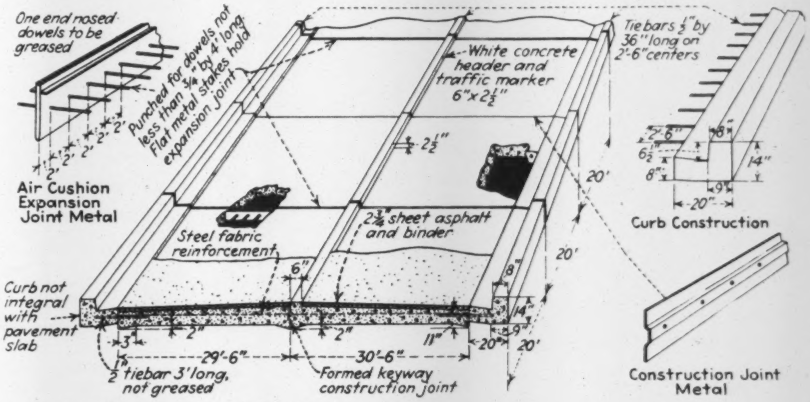
Consideration of the non-uniformity of the foundation quite definitely indicated that failures of the base concrete would probably occur at the junctures of these different subgrade conditions unless special provision was made to strengthen the slab. It was therefore important to design carefully and to construct this pavement base so that the slab would be maintained in a horizonal alignment, regardless of possible cracking causing separation of the concrete, and strengthened so as not to allow the development of free corners.

The paved roadway is 60 ft. wide. In order to facilitate the handling of traffic and not to necessitate closing the street, the pavement was designed and constructed in two widths of about 30 ft. each, a keyway being formed at their juncture to assist in maintaining a horizontal surface.

In the center of the 60-ft. roadway and integral with one slab is a white concrete header and traffic marker, 6 in. wide, 2½ in. above the base concrete and about flush with the top of the sheet-asphalt wearing surface.

The curb and gutter were integral but separate from the base slab. This curb and gutter section was, however, tied in with the slab by means of ½-in. tie-bars, 3 ft. long and spaced each 2½ ft.

As stated previously, it was believed that failure of the base concrete would occur at the juncture of the variations of subgrade conditions. The construction of a properly designed joint at these places only, to insure structural strength, would permit free movement of the concrete slab at these joints, and during periods of contraction of the concrete if would probably cause the development of a crack of considerable width in the bituminous surface. To guard against the development of cracks due to shrinkage during setting of the concrete, which, due to the free movement of the slab, would cause weakening of the pavement section, and to reduce the movement due to temperature variations at any joint to such a minimum that it was believed the bituminous wearing surface would not be disturbed, it was considered necessary to install joints at close intervals.


One end nosed dowels to be greased
Punched for dowels not less than 3/4" by 4' long
Flat metal stakes hold expansion joint
2'
2'
2'
2'
2'
Air Cushion Expansion Joint Metal
White concrete header and traffic marker 6"x2½"
Tie bars ½" by 36" long on 2'-6" centers
2½"
20'
2'-6"
8"
6½"
8"
14"
9"
20"
Curb Construction
Steel fabric reinforcement
2¾" sheet asphalt and binder
6"
20'
8"
Curb not integral with pavement slab
3"
2"
2"
11"
14"
9"
20"
20'
29'-6"
30'-6"
½" tiebar 3' long, not greased
Formed keyway construction joint
Construction Joint Metal

**Details of pavement base designed to keep horizontal plane and resist cracking at junctions where subgrade conditions change.**

The specifications of the District of Columbia highway department for concrete pavements require the installation of expansion joints spaced each 40 ft., so as to relieve internal stresses that may be caused by confining the concrete during the periods of expansion and to control cracking due to shrinkage during setting, and contraction due to temperature changes.

This design had not previously been used in pavement bases, but it was deemed advisable to follow the concrete pavement specifications for this project; and further to reduce the movement at the joints, contraction joints, formed by the installation of a metal strip placed the entire width of the slab, were constructed at mid-distance between the expansion joints. It is believed that this spacing or joints insures against any transverse cracking that might otherwise occur due to shrinkage during setting or to stresses set up by contraction due to rapidly lowering temperatures.

By the installation of the expansion and contraction joints it was possible to design and construct the slab against weak places, such as might result if cracks occurred at random, leaving free edges and corners.

This department has been using the metal air-cushion joint during the past two years for concrete-pavement expansion-joint installation, and due to their satisfactory service some consideration was given to their use in pavement bases. It is believed that this type of joint will prove particularly satisfactory for this purpose. Other types of joint material which are new in design and which are believed would prove satisfactory under such conditions are the cork and rubber joint materials. Data would indicate that joints constructed of any of the materials, metal air-cushion, cork or rubber would not extrude under compression sufficiently to cause excessive pushing up of the surface material.

In a further effort to reduce the movement of the concrete base, a minimum cement content was used. It was considered that as the amount of expansion and contraction is dependent on the unit cement content of the concrete, and that as it has been shown that the bituminous surface offers considerable protection to the concrete, and that the climatic conditions are not severe for the District of Columbia, this cement factor could be safely reduced from 1.6 bbl. per cubic yard of concrete, which is required for concrete pavements to insure durability, to 1.04 bbl. per cubic yard of concrete.

Dowel bars of ¾-in.-diameter steel rods, 4 ft. long, are spaced each 2 ft. across both the expansion and contraction joints. These bars were painted with red lead to protect the steel against corrosion at the joint, greased on one end to break the bond with the concrete, and properly nosed to allow free movement of the concrete. It is believed that the use of these bars, when properly installed, is an important factor in preventing vertical displacement of adjoining slabs and in transferring loads across the joint, thereby strengthening the pavement slab.

It has been established that for concrete pavement more than 20 ft. wide it is advisable to construct a center longitudinal joint so as to control the cracking that tests and service have shown could be expected to occur. However, it was decided to install mesh reinforcing of such weight (60 lb. per 100 sq.ft.) as would hold the concrete intact if such longitudinal cracks should occur. The longitudinal and transverse wires were of equal diameter.

This design divided the pavement into slabs 20 ft. long and 30 ft. wide. Both edges were thickened to 11 in., 2 in. greater depth than the center thickness. The outside edge is further strengthened by the curb and gutter section, and the inside edge by keying into the adjoining concrete section. The thickness of slab was determined from the corner design formula $S = \frac{3w}{d^2}$ used in the design of concrete pavements. That this formula is satisfactory for use in base design was shown by the results of the Bates road tests. The results of these tests show all pavements of rigid-type design are dependent for structural strength upon the strength of the concrete slab or base, the surface covering serving only as a wearing course and as a protection to the base against climatic changes.

The small V-section at the top of the air-cushion joint was poured with a filler made of rubber and rosin. It is believed a bituminous filler would work up into the asphalt surface but that the rubber-resin compound would not affect the surface composition.

This roadway is being watched very closely to study the value of the application of this design theory and if possible to determine a standard of construction for specification.

◆

# Winter Paving Provided Work in Ohio Village

**Winter paving in Ohio town started three supplementary operations, with payrolls in addition to its own, and positively demonstrated that simple precautions and high-early-strength concrete can whip the frost hazard**

**By Luke Brannon**

*Resident Division Deputy Director,*
*State Dept. of Highways, Middletown, Ohio*

LAST FALL the main street of Wyoming, Ohio, was 1¾ miles of worn-out village pavement in the excellent concrete main road between Cincinnati and Hamilton. The village wanted a new pavement. As the street was also a state and a federal road, arrangement was made to pave it with concrete and divide the cost between state, county and village in the proportions of 50, 30 and 20. The width of the pavement, 52 ft. for 1¼ miles and 41 ft. for ½ mile, made it too expensive for either body to assume full cost under their legal and financial limitations. The arrangement also was that the state highway department should plan the pavement and direct construction.

The work was planned as a winter unemployment relief measure. The state share in the cost was taken from its allotment of federal emergency-relief money, and all conditions of the federal emergency relief act were complied with as to employment, hand labor and hours of work. The work was let to contract, and construction was begun in October, 1932 As carried out, the outstanding features were the spread of employment and the methods employed in cold-weather concrete paving.

Boiler on skids, hauled behind paving mixer, supplied water to heat concrete for cold-weather paving.

## Spread of employment

Welfare relief labor was employed on the work at minimum wage rates of 45c. an hour for unskilled and 67½c. an hour for skilled labor. Employment of any laborer was limited to a 30-hour week, and the Bureau of Public Roads' regulations for hand operations were adhered to. On the paving the records cover 23 men supervising, 82 truck drivers, 77 skilled and 478 unskilled laborers from Oct. 28, 1932, to Feb. 4, 1933. They show 660 men employed at wages amounting to $11,281. To supply the aggregates, two gravel producers employed 14 men for 24 to 27 days at wages of $1,413.

Coordinate operations were as follows: The new paving required the village of Wyoming to relocate, replace and construct street and under-street structures at a cost of $972 for materials and of $1,698 for labor. It also constructed a stormwater sewer to handle the road drainage at a cost of $8,012 for materials and $16,225 for labor. Lowering the gas main by the gas company cost $13,000 for materials and $14,700 for labor. All these coordinate operations were conducted as unemployment relief and utilized as much hand labor as possible. Together they paid out $32,623 in wages in a period of four months.

## Winter paving methods

Removal of the old pavement and preparing the subgrade involved no unusual methods, and the paving concrete conformed to standard state practice. The special practices developed for winter operations were as follows:

Aggregates were batched at the gravel plants 3½ miles away from bins equipped with steam coils and were trucked to a 27-E mixer, passing a railroad siding where cement was added. The trucks were provided with batch partitions and with canvas tarpaulins to cover and protect the cement. The mixer was placed on one-half of the old paved road alongside of and parallel with center steel forms. A steam boiler mounted on a sled was placed immediately in the rear of and cabled to the mixer, being moved forward with the mixer plant as the paving operations progressed. The water line from the fire hydrant led to the boiler and from the boiler to the mixer, so that hot water was available to assist in heating the concrete at such times as needed. Except for a few times, when outside temperatures rose, hot water was continuously used.

The average temperature of concrete as laid on the subgrade ranged from 60 to 65 deg., but at times it reached as high as 75 deg.

To meet winter weather conditions and avoid delays in starting the concrete on mornings after a very cold night, the prepared subgrade for a reasonable distance ahead of the complete pavement slab was protected with a heavy covering of straw. The following morning the straw was sprinkled with gasoline and burned, thereby thawing and removing any light frost and warming the forms so as to permit an early start.

The concrete having been deposited and properly graded, the surface finishing was completed by a finishing machine. This operation was immediately followed by a 12-ft. longitudinal float manipulated by two men from two bridges that spanned the pavement. Such a float is operated parallel with the center line; beginning at one side of the slab, it is pulled toward the other side with a wiping motion. The float erases any ridges at right angles to the direction of traffic, and is a most efficient tool for securing a smooth-riding surface. It consists of a plank from 12 to 20 ft. long, stiffened with a rib along the top and provided with plow handles at each end.

Following the longitudinal float, a finisher (with a 10-ft. straight edge to check the surface for smoothness and deviation) operated a long-handled float to remove irregularities, water and laitance. For final finish, a very rough folded strip of burlap, slid lightly over the surface, gave the pavement a gritty surface against skidding.

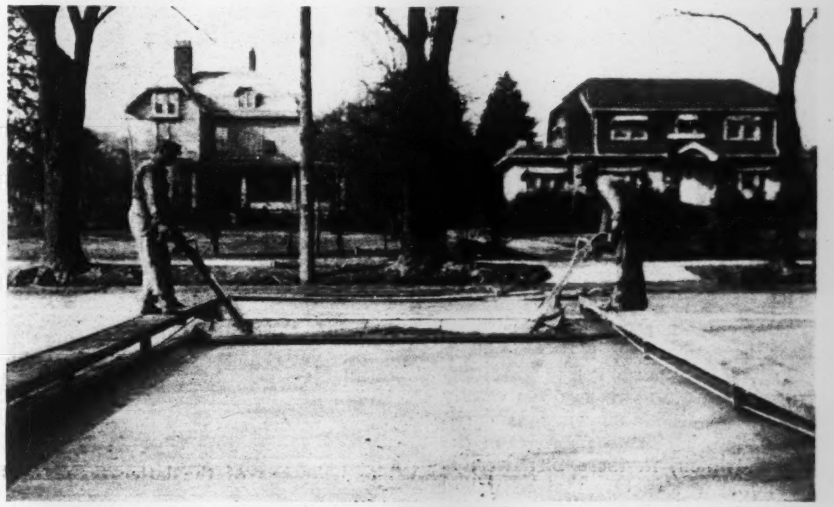
To retain heat in the poured concrete, the pavement was covered with burlap and straw as soon at it had set hard enough not to become damaged by the weight of the covering. It is interesting to note that when a 60 deg. concrete was placed during atmospheric temperature of approximately 35 deg. and the slab was properly covered and protected, a thermometer placed under the covering the next morning registered 60 deg. when the outside temperature had dropped to 20 deg. or less during the night.

The usual procedure for checking irregularities of surface is to remove the covering the following morning and to check with a bumpometer. In this instance it was decided to be more essential for the pavement to retain its heat, which assisted in the development of high-early strength, at the expense of surface grinding.

The contractor, with the approval of the state highway officials, selected high-early-strength cement, and tests by the state testing laboratory, Columbus, Ohio, were made with such good results that the continued use of this particular brand of cement was sanctioned.

### Tests for future use

This project being one of the largest paving contracts in the United States using high-early-strength cement, it was deemed advisable to make thorough and systematic tests for future reference. The testing laboratory sent an engineer to supervise the making, curing and breaking of cylinders. Test breaks were made at 24, 48 and 72 hours, and at seven-day intervals. In conjunction with the tests on cylinders and beams, cores were taken at 24-, 48- and 72-hour intervals, the 24-hour cores being crushed locally, while the 48- and 72-hour cores were shipped to the laboratory at Columbus, Ohio. Coring proved an effective method of checking both the depth, quality and compressive strength of the pavement slab, especially since the slab was constructed under and was then subjected to very severe weather and temperature conditions.



Heavy plank float pulled in a cross-street direction with a wiping motion removed finisher ridges.

The average compressive strengths of cores and 6x6x42-in. beams were as follows:

| Age, Hours | Cores, Lb. | Beams, Lb. |
|---|---|---|
| 24 | 1,716 | 235 |
| 48 | 2,858 | 384 |
| 72 | 3,760 | 534 |
| 7 days | 4,630 | 750 |

The slump tests taken in conjunction with the sample compression cylinders showed slumps ranging from 1¾ to 2½ in. This range of slump was due to variation in the moisture of the batches. Test cylinders, 6x12 in., showed the following average compressive strengths:

| | |
|---|---|
| 2 days | 3,576 lb. |
| 3 days | 3,677 " |
| 4 days | 4,916 " |

In enumerating the purposes for which high-early-strength cement was selected, the most noticeable results were: reduction of the curing period, elimination of detours at intersections, opening the pavement for traffic at a much earlier date, permitting property owners to use driveways in a shorter period, lessening the inconvenience to traffic and population served and enabling the highway department to carry out an emergency construction program, whereby employment was given to many of the unemployed in Hamilton County during the fall and winter months.

### Personnel

The construction was carried out under the direction and supervision of Luke Brannon, division engineer, Robert Fisher, assistant division engineer, Walter E. Willard, resident engineer of Hamilton County, and Henry Nunlist, field engineer. These men are all state highway engineers. The De Salvo Construction Co., Cincinnati, Ohio, contractors, maintained a field office in Wyoming, the work being supervised by Russel De Salvo.

## Wisconsin Sewerage Status

Sewage from 118 out of 232 municipalities in Wisconsin is cared for through public sewerage systems. It is treated in 125 plants. Five new systems and a number of treatment plants were installed in 1932. Janesville, Fort Atkinson and Edgerton are having plans prepared on intercepters and treatment works. Jefferson has secured plans. These developments are part of the Rock River clean-up program. Altoona is having plans drawn, and Hartland expects to install a sewerage system that will include treatment works. The Green Bay Metropolitan Sewerage District has had the constitutionality of the district law affirmed by the state supreme court, and improvements to abate the serious pollution of the East and Fox rivers will be started in 1933. Further consideration will now be given to a metropolitan district covering the Fox River from Neenah to Kaukauna. Madison has such a district formed under the general law. The Milwaukee district is under a special law.

# Long Beach Earthquake Emphasizes Known Facts

**Although teaching a few new lessons, the recent California earthquake largely substantiated existing knowledge and concentrated its damage on structures lacking unity and strength and the benefit of sound structural design**

By Walter L. Huber
*Consulting Engineer,
San Francisco, Calif.*

THE EARTHQUAKE which damaged or destroyed many structures in the region of Long Beach, Compton and portions of Los Angeles on March 10, 1933, brings out a few new lessons, but more particularly does it emphasize the lessons derived from other earthquakes.

It is not surprising that practically all earthquake reports are to some extent exaggerated. This is often wholly unintentional. The tendency is, however, to describe or to photograph the abnormal which, in these instances, is the building or structure that has failed or has suffered perceptibly. Its neighbor next door, which may have been built in accordance with sound engineering principles and thereby escaped practically unharmed, gives the same appearance as it did last week or even last year and is therefore not the subject of much interest at the moment, particularly to the lay observer or photographer. Thus, a record is left which is more or less complete so far as damage is concerned, but very incomplete in respect to the well-built structures that were unharmed.

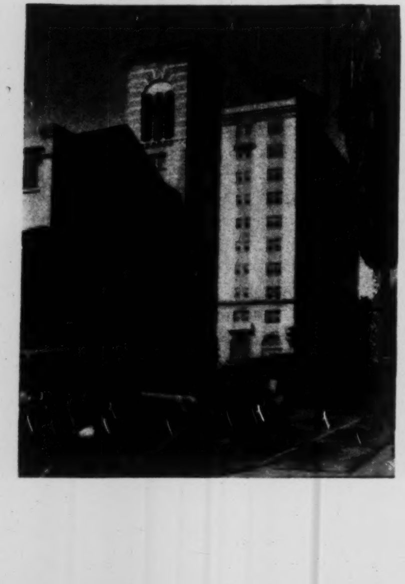
The performance of well-built buildings of both steel and reinforced-concrete construction, ranging in height up to some fifteen stories, is very reassuring. Instances of these are shown in the illustrations of the Villa Riviera Apartments, the Security First National Bank Building and the Heartwell Building (*ENR*, March 23, 1933, p. 378), which were, when photographed a few days after the earthquake, but little the worse for their experience. Many other examples could be cited.

The condition of these buildings should be compared with those constructed of ordinary light brick bearing walls supporting floor and roof joists. While buildings of the latter type are not dependable for resistance to earthquake stresses, while they have furnished the most spectacular examples of complete wreckage, and while I do not advocate this type, I do not wish to leave the impression that their complete destruction was by any means general. There are many examples of strongly built masonry walls that survived with little damage. An example is the church tower and facade shown in Fig. 1, although it should be noted that the corresponding brick gable in the rear wall of this church building fell. Other examples could be supplied in great numbers.

### Brick-wall failures

The inability of many brick-wall buildings to resist the stresses imposed by earthquakes lies in poor workmanship and improper materials. This point is well illustrated in Fig. 2, a view of a partly collapsed lodge building in Compton. Here is a 13-in. brick wall with pilasters upon which roof trusses are supported. Header courses are quite unknown; whole stories are laid up without them. Often mortar does not fill the vertical joints. To these conditions, so evident in the photograph, the reader needs only to add that the mortar itself is in most instances a lime mortar with little adhesive quality, which can easily be powdered in the hand, and he will have a complete picture of the reasons for failure of these walls under earthquake

**Fig. 1—Substantial construction resisted the earthquake. The brick church and tower were little damaged except for the loss of a rear gable. The Willmore Hotel, beyond, was also of sound construction and suffered only slight damage.**



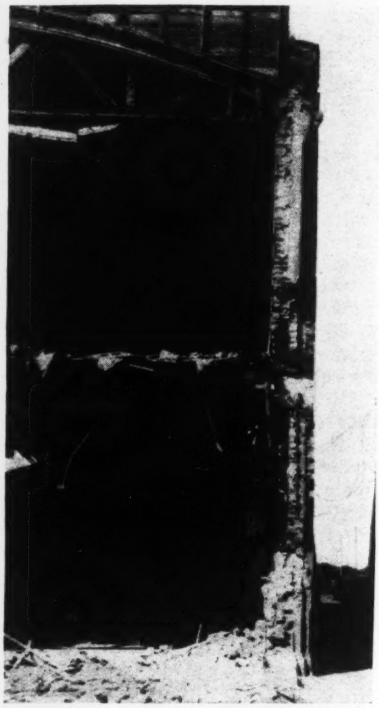
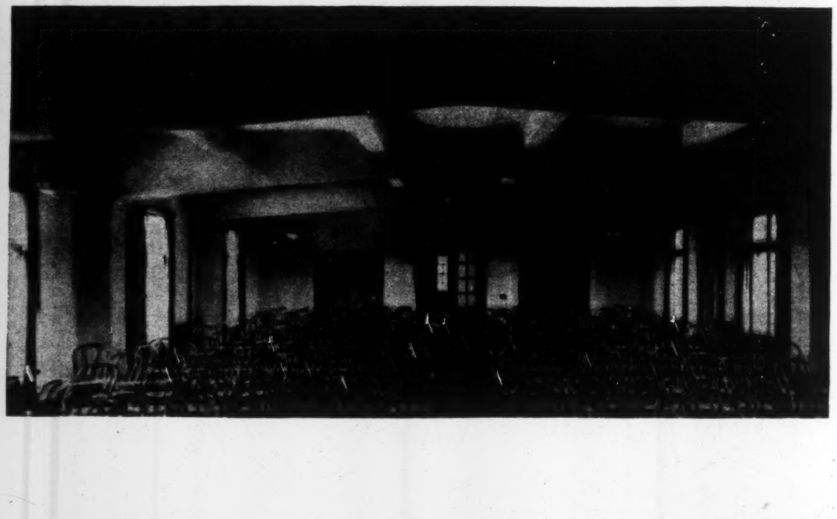
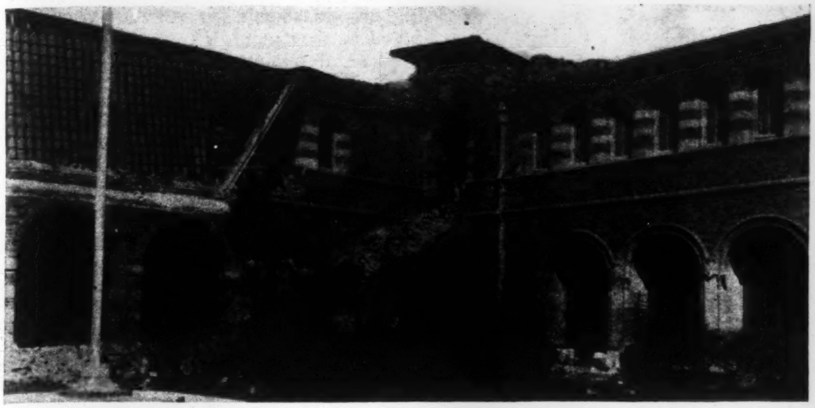


**Fig. 2—Poor bond of masonry revealed in a lodge building in Compton was typical of many buildings in the earthquake area.**

**Fig. 3 — Uninjured reinforced-concrete rigid-frame construction in which the columns were heavy enough to resist the girder movements.**





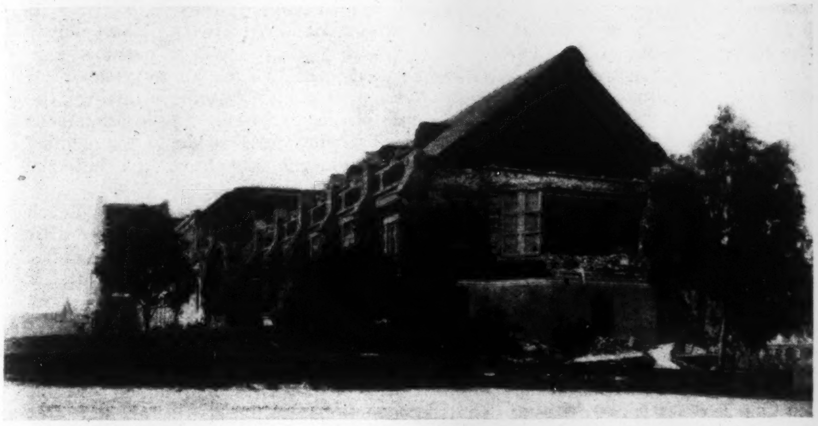


**Fig. 4—Bad conditions at two Long Beach schools. Above, Alexander Hamilton School, where the tower and part of the second-story wall fell on exit stairs and walks. Below, John Muir School, where debris blocked the exits.**

stresses. In the design of these buildings probably no thought was given to the resistance of lateral forces due to earthquake. The contrast between this masonry and that which I examined a few years ago at Charleston, S. C., and which had survived the Charleston earthquake of 1886, is marked indeed. Certainly the art of masonry construction has deteriorated under the urge of competitive speculative building. The builders of the splendid masonry construction of early Charleston would be horrified at present-day practices. I would not, however, limit my criticism of present brick masonry construction to that of Los Angeles County. Similar construction is now found in practically every American city and only needs a catastrophe to expose it.

### Destruction of schools

Unfortunately, the schools throughout the afflicted area were very generally constructed of inferior masonry bearing walls supporting long-span timber joists and roof trusses. The almost complete destruction of many of the school buildings is one of the outstanding developments. Had the earthquake occurred during school hours the loss of life and injuries would have been appalling. It is to be hoped that from the lesson derived better construction of school buildings and places of public assembly will result. The public memory is short, however, and is often dulled by false ideas of economy. The fault here lies with the administrative officials who, knowingly or unknowingly, choose this type of construction for public buildings; it lies also with those who follow, if they do not actually direct, the program by preparing the plans; and with the builders who in many instances have been responsible for poor workmanship and improper materials. Again, these faults of school buildings are not peculiar to Los Angeles County—they are very general in other localities.

After the Santa Barbara earthquake of 1925 a study of reinforced-concrete-frame buildings disclosed many instances where concrete girders of long span proportioned for heavy bending moments were supported on comparatively small columns designed only to resist the vertical loads of low buildings. The two members were constructed monolithically. The stresses induced by the earthquake caused the heavy girders to rupture the lighter concrete columns. Instances of this kind are not so evident at Long Beach, although they may be disclosed in connection with some of the repairs.

In the Y.W.C.A. Building certain long-span concrete girders were designed to act with their supporting columns as rigid frames (Fig. 3). Although subjected to some of the most violent effects of the shocks, this construction shows no damage whatever. The damage to this building, which has brick walls supported on a concrete frame, is limited to the displacement of certain bricks where a 4-in. layer is carried up past a concrete spandrel beam at the roof level and to certain other cracking of face bricks. It is of interest to note that on this building brick parapet walls were constructed by carrying the columns above the levels of certain roofs and decks and joining them by a 6x4x$\frac{1}{2}$-in. angle carried in the top of the brick walls. These parapets are intact, as may be seen in Fig. 5.

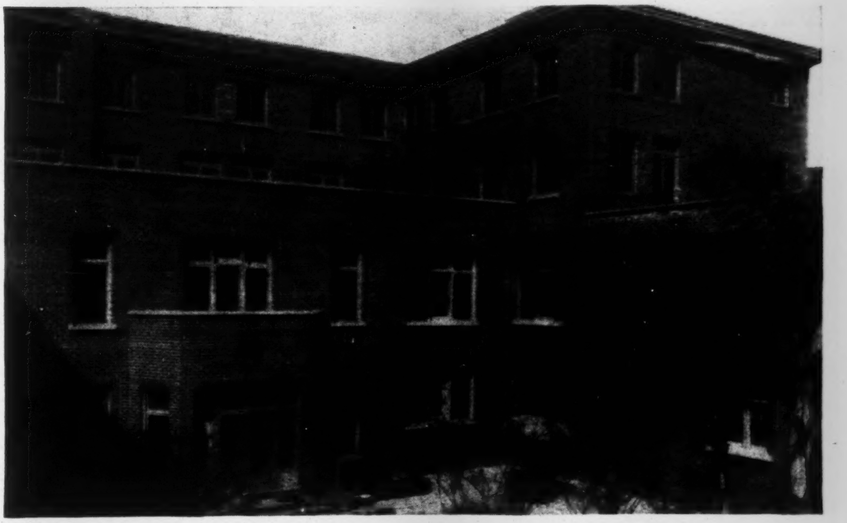


**Fig. 5—Brick parapets of Y.W.C.A. Building, stayed by a steel angle framed between the tops of the columns, remained intact.**

# High Seismic Factors in Recent Earthquake

**Rough calculations on damaged tension members in some structures indicate possible horizontal acceleration of 0.2 *g* or more**

By L. H. Nishkian
*Consulting Engineer, San Francisco, Calif.*

IMMEDIATELY after the Long Beach earthquake of March 10, O. C. Struthers, Pacific Coast manager of the Raymond Concrete Pile Co., made a survey of some of the wrecked structures and later asked L. H. Nishkian, consulting engineer, San Francisco, to supplement his observations and prepare a report on the action of pile foundations. Mr. Nishkian, after a visit to the earthquake area, prepared the report, which not only covered foundations but contained some interesting conclusions concerning the probable acceleration of the earthquake. By calculating the force required to cause the damage to some X-bracing members in several different structures, he concluded that seismic factors of 20 to 25 per cent must have been present. The conclusion is particularly interesting in view of the relatively high accelerations recorded by the strong-motion accelerographs of the Coast and Geodetic Survey, as reported in these pages last week. The following information represents Mr. Nishkian's report except for some of his more general comments and for those portions of it referring to facts already recorded in our issues of March 16 and March 23. —EDITOR.

JUDGING from the damage done and the extent of the seriously damaged zone, the Long Beach earthquake was of less severity than the San Francisco shock of 1906 and, perhaps, less than the Santa Barbara shock of 1925—probably about IX on the Rossi-Forel scale. From some indications and the reports of those who felt it, the shock was quite severe initially, but of short duration.

## Pile foundations

As was brought out in the Japanese earthquake at Tokyo, buildings on pile foundations showed remarkably better than similar buildings on other foundations. At Compton, probably the most heavily damaged zone, a two-story school building on piles suffered far less than a one-story adjoining unit not on piles. At South Gate there were two very similar 150,000-gal. steel water tanks on high steel towers about three blocks apart, one on poured-in-place concrete footings tied together and resting on piles and the other on spread footings. The tank on spread footings collapsed, while the one on piles, which was full of water at the time, stood up, although the X-bracing was stretched and some of the columns showed lateral deformation. The Kress building at Compton is a brick building on piles and was probably the least damaged in that district, the principal damage being some cracks in the rear wall. The Municipal Auditorium at Long Beach, a large monumental-type structure near the beach, carried on piles, appeared to be in excellent condition, although the ground adjacent settled as much as a foot in some places.

Two reasons appear to the writer for this good behavior of buildings on pile foundations. It is known that an earthquake acts more severely at the surface than at some distance below, perhaps due to surface undulations in soft material. Piles probably protect a building from such undulation. Secondly, and perhaps the more important factor, is the bending that the piles can sustain, thus materially relieving the building from the full shock. In this respect piles are analogous to a flexible link between the ground and the building proper.

## Probable seismic factors

At Seal Beach, a few miles south of Long Beach, a large generating station has a concrete stack 300 ft. high, 32 ft. in diameter at the base and 24 ft. at the top. The stack rests on a heavy concrete slab supported on steel girders and eight steel columns that carry the load to a foundation about 75 ft. below, making a total height of about 375 ft. from the ground or foundation level. The eight columns are arranged in groups of four. The upper 40 or 45 ft. in the north-south direction is X-braced two panels high, and the lower 30 ft. is braced by portal framing. Each member of the X-bracing consists of two angles 6x4x⅜ in. These angles had generally buckled, and the paint at the end gusset plates had wrinkled, indicating that the angles had been stressed nearly to their ultimate strength. A rough calculation of this stress and of the weights involved indicated a seismic factor (ratio of horizontal acceleration to acceleration due to gravity) of 15 to 20 per cent. The anchor bolts at the base of the south steel columns had stretched or slipped about ½ in. in some cases. On the roof alongside the stack were several steel water tanks, of about 15,000 gal. capacity, supported on vertical, X-braced steel columns. Two tank towers were two panels high, one tank tower was one panel high, and all tanks were full of water. The X-bracing consisted of 1-in. round bars, not upset. Many of these bars in the upper panels were broken or excessively stretched where threaded. A rough calculation indicated a seismic factor of 20 to 25 per cent, although this may have been higher, inasmuch as the X-bracing failed at that point.

The interesting observation to be made from the above is that in spite of this very high seismic factor the tanks and framing stood up. After the rods failed, the structure was flexible, and further damage was arrested. The column groups supporting the stack acted in a similar way. The lower seismic factor in the stack framing may be accounted for by the fact that the lower 30-ft. section was partially flexible and thus relieved the more rigid X-bracing above. However, there are too many factors involved to make this explanation of the behavior of the structure entirely conclusive.

In the tank that was one panel high, the rod on one side only failed, causing the four columns to twist. The collapse of some tanks elsewhere perhaps may be attributed to such an unsymmetrical failure of the X-bracing, raising the question as to whether some other type of bracing not subject to sudden failure might not have averted the collapse.

The Armory Building at Long Beach is a reinforced-concrete structure that suffered only a few unimportant cracks. One large room about 65x85 ft. is spanned by steel roof trusses with plank decking. Two panels of this roof were X-braced in the planes of the top chords with 1-in. round rods, not upset. Some of the rods had pulled out the connections to the truss at the side walls. Failure involved the shearing value of one ¾-in. rivet. A rough calculation indicates a seismic factor of 25 per cent or more. No further damages had resulted after the failure of the X-bracing.

Many steel water tanks on high steel towers were still standing after the earthquake. In some cases the X-bracing rods had failed; in all others observed by the writer, the rods had been stretched and were sagging, the greatest sag being in all cases in the top panel immediately below the tank. This draws attention to the difference between earthquake and wind stresses. It is quite simple to assume a horizontal force and to design the tower for that load, but earthquake stresses do not behave that way. The factor of time must be taken into consideration, either as the time required for the induced stresses to pass from point to point, or as expressed in the period of the structure.

If the high seismic factors indicated above are verified, it would seem practically impossible to design tall buildings for such high lateral loads. However, it is becoming increasingly evident that other factors than the horizontal acceleration and the static resistance of the structure must be considered. In addition to more accurate information on the seismic factor, broader knowledge of period, amplitude and direction of earthquake movements must be secured.

# Letters to the Editor

## Value of Reinforcement in Concrete Columns

Sir—In a recent article (*ENR*, Sept. 8, 1932, p. 277) T. Germundsson pointed out possibilities in reducing the size of columns through the use of heavy longitudinal reinforcement connected at floor levels by welded splices. Quite a different view was expressed by Dr.-Ing. M. Hajnal-Konyi (*ENR*, Feb. 23, 1933, p. 257), who argued, by reference to the 1932 specifications of the German committee for reinforced concrete, that it would be economical to limit the longitudinal reinforcement to a small percentage and to use a large amount of hooping (6½ per cent in one example that is cited).

By the German formula the design load is evidently one-third the ultimate strength. The latter is expressed as equal to the combined strength of concrete core section, the longitudinal steel at its yield-point stress, and the hooping steel at its yield-point times an effectiveness factor, 2.5. Such an expression has been generally verified, except that American tests have shown a value of 2.0 instead of 2.5; this value was for a length-diameter ratio of 7.5 and would decrease rapidly as $l/d$ increased. With $l/d$ as great as 20, as permitted by the German ruling, I doubt if the effectiveness of the hooping is more than a small fraction of the value 2.0.

The German ruling apparently neglects two pertinent features of column action: the deformation due to time yield and the gross deformation beyond the yield point of a column. (The column yield point may be defined as the load at which the yield point of the vertical steel and the ultimate strength of the plain concrete in both core and shell are reached). Beyond this yield point the shell spalls off, the column shortens and bulges greatly, the hooping finally receives stress and, by restraining the concrete core, contributes strength to the column. The accompanying shortening may be ½ to 3 in. in a story height. Opinions differ greatly on this question: How much of the column strength developed beyond the yield point, and attended by gross deformations and spalling, should be permitted to receive major consideration in a design formula? The case is analogous to the margin of strength of structural steel beyond its yield point, where ductility rather than strength is of most value.

Aside from contributing strength, the spiral reinforcement insures reliability of action and the ability to give warning of and to produce a gradual failure (as contrasted with the sudden failure of tied columns). For this reason some premium may be placed on the use of spiral reinforcement, and the writer believes that this should be accomplished not by the use of a "Considere-type" formula, expressing the last ounce of resistance that may be squeezed from the column, but by the judicious choice of the factor of safety. This was the method followed in the 1917 joint committee report, wherein a flat 1 per cent of hooping was specified and recognized, not in the equations, but in the permissible stress on the concrete; it was also utilized in the recent report of the A.C.I. column committee 105, wherein certain spiral requirements are made but not recognized in design equations, except as reflected in the factor of safety used.

As a specific example, consider Hajnal-Konyi's proposal of a column of 2,500-lb. concrete, 24 in. square, with 20-in. circular core, a vertical steel percentage of 2.16 and a hooping percentage of 6.48. A hooping percentage thrice the vertical is the maximum and economically the most favorable permitted by the German rulings. With the steel stresses cited in the letter, the ultimate strength of this column (on core area) should be 10,780 lb. per sq.in., the design stress 3,590 lb. per sq.in., and the yield point 5,280 lb. per sq.in. While the ultimate factor of safety is 3, the factor based on yield point is only 1.47. If the column had been round instead of square, the yield point would have been 4,330 lb. per sq.in. and the corresponding factor of safety 1.21. For these two columns a design load would produce initial stresses of 1,700 and 2,070 lb. per sq.in. in concrete (gross area) and 23,100 and 28,200 lb. per sq.in. in vertical steel. From recent data on time yield effects we may estimate that a few months of sustained design loading would stress the vertical steel beyond its yield point. This need not impair the strength of the column, unless the value of $l/d$ was high, but there would surely be an indeterminate and undesirable shortening.

Comprehensive studies of time yield show that its effects decrease as the amount of vertical steel is increased. This indicates that, to avoid undesirable column shortening, the factor of safety should be increased when small amounts of vertical steel are used. Such a variable factor of safety is embodied in the 1924 joint committee formula for spiral columns and is perhaps the outstanding feature of that formula.

The conflicting views of Germundsson and Hajnal-Konyi, advocating on the one hand a stiff elastic column with heavy vertical and nominal spiral reinforcement, and on the other hand, a member that becomes highly compressible at loads not greatly above design values due to the use of heavy spirals, are presented opportunely, since new formulas are now being considered which embody to some extent one or the other of these viewpoints. In my opinion the most economical design under the German ruling will produce a very unsatisfactory type of column.

Regarding the calculations as to economical design, may I add that if the relative effectiveness of spirals is reduced from 2.5 to 2.0, or less for slender columns; if steel of similar quality is used for both kinds of reinforcement, as is the case in much of American design; and if the large extra cost of spirals is given consideration, a great deal of the oft-quoted economic advantage of spiral reinforcement vanishes. What remains must be weighed carefully against the fundamental disadvantages attending the use of heavy spiral reinforcement. F. E. Richart.

Urbana, Ill.,
March 17, 1933.

Research Professor of
Engineering Materials,
University of Illinois.

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## Cross-Connection Experience in Wisconsin

THE LAST typhoid epidemic in Wisconsin attributed to cross-connections between safe and unsafe water-supply systems was in 1929. Later a state-wide program was developed by the state board of health to secure proper protection of such connections or their elimination. In the case of large auxiliary supplies employed for fire protection, double check valves of approved design, equipped with drip cocks, gages and gate valves for testing, have been required, with monthly inspections by the waterworks and industrial-plant officials. In the cases of industries or commercial establishments desiring to use the public supply in addition to a private supply, elevated tanks have been used with the public supply entering at the top above the overflow line. Otherwise, protection is accomplished by swivel connection with elbows, unions and gate valves, by means of which the water from either the city system or private source may be used but not both at once. Where the private supply is of good sanitary quality connections have been permitted only when arrangements are made for periodic analyses.

At the end of 1932, according to the last quarterly progress report of Charles A. Halbert, state chief engineer, the status of cross-connections in the state was as follows: 122 out of 305 municipalities report that such connections exist, with a total of 341 connections. Inspection has been made of 336. Of the 189 non-potable cross-connected supplies reported, 97 received permits subject to proper maintenance, 79 were eliminated, and 13 were unsatisfactory at the last inspection. Of the 152 potable cross-connected supplies, 107 were permitted subject to proper maintenances and safe analyses, 22 were eliminated, 12 have permission pending, 6 were unsatisfactory, and 35 were held up pending inspection.

# Engineering News-Record

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Editorial and Publishing Offices at 330 West 42d St., New York; 520 North Michigan Ave., Chicago; 883 Mission St., San Francisco; Aldwych House, London, England.

Subscription price $5 per year in the United States; foreign rates on application.

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April 13, 1933 Volume 110 No. 15

## *Striking at Technical Services*

REPORTS from many parts of the country tell of the reduction or wiping out of engineering services. In part they merely repeat what has become familiar during the past year or two, but they indicate that the movement to reduce technical services is now more active than ever. Governmental as well as corporate organizations are cutting technical services severely in their drive for expense reduction. Recent figures from a particular county, for example, show that nearly all of the reduction in county personnel is being made in the departments occupied with inspection, road maintenance and engineering, while newer and less essential departments remain virtually untouched. The injury that such discrimination does to the public service is intensified by the continued effort to reduce public-works construction, most strongly so in the highway field. This latter effort is not honest, for the road tolls that are collected as a specific charge to pay for road improvement continue to be collected but are diverted to other uses—a public fraud that is already beginning to develop the inevitable reaction. Yet, whether honest or dishonest, the movement calls for protest. It is producing further engineering unemployment at a time when the greatest responsibility of public officials is to combat unemployment by creating useful wealth.

## *A Time to Buy*

A SPECIFIC instance of how the public can buy large values at low cost is found in the surveying and mapping operations of the government. The woeful incompleteness of the existing surveys is well known, as is also the fact that public and private operations of many kinds are subjected to excessive costs and delay thereby. There is no room for doubt as to the need for concentrated attack on the survey problem, nor any question as to the economy of doing it at the unusually low costs that can be realized today. The paramount consideration, however, is what the work will do against unemployment. C. H. Birdseye outlines this effect in the present issue, adding timely information to what was said in our issue of March 30, 1933, p. 417. He recalls how the progressive efforts of a group of public-spirited engineers last year resulted in inaugurating much survey work and provided a large amount of employment. The same thing can be done in many other states, at small outlay. The water resources work of the Geological Survey has need for 15,000 man-months of essential work, another large item is available in land-classification surveys, and more men can be used to advantage in the work of the Coast Survey. Activity in all these operations has been curtailed, and further curtailment is indicated. Instead they should be increased. Such increase is a necessary part of the public-works program now under consideration, and engineers should not overlook the fact in giving support to that program.

## *Unknown Storm Dangers*

IN the destruction of the great airship "Akron" last week, strangely like that of the "Shenandoah" eight years ago, we have a new and tragic reminder of human ignorance. The limited knowledge of wind action that suffices in bridge and building design, where large load forces are involved, fails in the case of the extreme requirements of a structure that must be lighter than air. The present disaster therefore cannot be likened in its nature to a bridge or a building failure. Both the "Akron" and the "Shenandoah" were wrecked in air currents that first made them uncontrollable, then put their controlling devices out of action, and, it appears, finally destroyed the integrity of their framework; so far as known neither involved a structural defect or a failure of the framework to resist the forces for which it was designed. The "Akron" was stronger than its predecessor, and its pressure equalizing provisions were so far superior that it should have been able to withstand the rapid rising and falling that presumably overstressed the "Shenandoah's" frame; yet the ship was wrecked by wind violence despite these precautions. It would seem that present metallurgical and aeronautical knowledge is not sufficient to make such vessels able to survive all hazards, and that their safety depends on good fortune coupled with that farsighted judgment which enables one skipper to weather a storm where another would be lost. But progress is certain, and the future will bring stronger airships if lighter-than-air construction is not wholly abandoned.

## *Reasonable Preparedness*

SOUND recommendations for future building procedure in Southern California are contained in the verdict of the coroner's jury of construction experts which reported on the Long Beach earthquake. Although not differing from the conclusions that might have been reached by any other investigating group, the result will have greater weight because the public will base its opinions on the findings of this official body and thus establish a starting point for much of the coming permanent reconstruction. The jury urged that Los Angeles County adopt the Pacific Coast Building Officials' code; it recommended that building design and construction be done only by registered structural engineers or licensed architects who qualify as structural engineers, and it stated that all school buildings more than one story high should be of Class A construction in steel or reinforced concrete. These specific recommendations are summed up by a plea for reasonable preparedness. The wording of this general recommendation is as appropriate as its substance. Many persons have assumed that earthquake-resistant design is necessarily "unreasonable," particularly in cost. The Japanese banks whose designs are described in this issue had their cost increased only 10 per cent by the provisions made against earthquakes, which is not an unreasonable charge for protection. These buildings incidentally exemplify the simple nature of some of the essential provisions, such as integration of foundation, frame and walls, the use of reinforced walls and partitions and positive anchoring of face stones cornices and decorative attachments. A building is not a pile of elements but a unit, and it is wholly reasonable and absolutely necessary to consider it as such in any region known to be subject to earthquakes. If the California cities will follow a program of reasonable preparedness in building new structures and in strengthening existing ones, earthquake damage can be limited to thousands instead of millions of dollars. And no one will be killed by falling walls and cornices.

## *Frost Hazard Discounted*

OHIO road construction, as described in this issue, besides indicating possibilities of off-season employment, is a record of definite progress in concrete roadbuilding in winter. True, the temperatures were not low compared with the seasonal frigidity of more northern zones, but they were lower than those that concrete road specifications commonly set as the limit for concreting. Very simple precautions served to check the frost hazard. Warm aggregates, heated mixing water and straw blanketing over the pavement slab accomplished the task, with the aid of a high-early-strength cement to reduce the period of danger from exposure. These preventives of frost damage naturally added to the cost, but the cost increment was outweighed by the advantage of carrying on construction without interference by the peak traffic of summer. Aside from its effectiveness in relieving local unemployment, the operation described has value in allaying the common fear of trouble from cold-weather concreting and therefore will assist in extending the range of possibilities for winter roadbuilding.

## *Better Concrete-Frame Design*

THE TIME has passed when a reinforced-concrete frame was considered practicable only for those buildings in which column spacing was uniform and in which, therefore, code coefficients for positive and negative bending moments for the beams would apply. The change came about through increased knowledge of continuous-beam design, which showed that the use of moment coefficients was often uneconomical and sometimes unsafe. At best, these coefficients were stopgaps, devices to avoid what was regarded as tedious and unnecessary computation, while in their worst aspect they were merely substitutes for knowledge. An excellent example of a structure that benefited from modern concrete-design principles is the Flagg Court apartment building in Brooklyn, described in this issue. The designer, in this instance, had no choice in laying out an ideal frame from a design standpoint; the ideal set was convenience and economy for the tenant, and the designer's job was to fit his frame to this ideal. He chose to base his design on the principles of the elastic theory, using only the fundamental equations for the relations between load, shear, moment and deflection. Moment coefficients were given no consideration. The resulting frame is slender but safe, since it utilizes the steel and concrete economically. Numerous other methods of theoretical analysis might have been used—three-moment, slope deflection, conjugate points, influence lines—but, in any event, facts as they exist in a reinforced-concrete frame were faced. The moment coefficient is not obsolete, for it often is a useful expedient; but a higher order of design intelligence throughout the profession will relegate it to its proper place, and better concrete-frame design will result.

## Chemical Sewage Treatment

CHEMICAL treatment methods for the disposal of sewage have greatly attracted the interest of sanitary engineers and sewage-works operators within the past year or so. A truly remarkable development is this revival of interest in one of the oldest forms of sewage treatment.

The use of a chemical coagulant to promote sedimentation of sewage arose in England nearly seventy years ago and achieved its greatest popularity between 1880 and 1890, when many chemical treatment plants were installed. It is remarkable to read the history of that early period and discover that practically every coagulant now being urged for use in sewage treatment was tried out. But it was found that large amounts of costly chemicals were required, quantities of sludge were produced far beyond the ability of equipment then available to handle economically, and the treated effluent, though clear, too frequently began to putrefy shortly after leaving the plant. When the trickling filter became available for use, together with methods for the septic disposal of sludge, chemical precipitation fell into disuse.

But the fact that chemical methods were tried out thoroughly many years ago and found wanting does not by any means demonstrate that failure is foredoomed for chemical methods today. Sewage treatment itself has made many advances during the years when chemical treatment was in disuse. Chemical and biological behavior is far better understood and mechanical equipment of far greater capabilities is now available. The chemistry of coagulation has become a nearly exact science. Equipment far different from the bag filter presses of the older days is now capable of handling inoffensively and economically the sludge produced.

Most important, however, is the possibility of using chemical processes as adjuncts to the established biological methods, to the end that the combination will be capable of giving better results and greater economy than either process alone. Properly speaking, the two are not today in direct competition but are complementary. Indeed, the most successful recent applications of chemical methods have been made in conjunction with existing plants of the biological type. At Plainfield, N. J., for instance, the use of a chemical coagulant has resulted in greatly improved operation of a trickling filter plant. The use of lime in connection with sludge digestion has for some years been accorded a place in established operating practice. Chlorination for control of odors in an orthodox sewage plant has been used in a number of places with complete success. The use of chemicals represents a valuable new tool to the designer and operator of sewage plants.

A study of the past history of chemical precipitation, however, makes available one lesson which cannot be safely ignored today. Sewage purification at that time was largely wrapped in mystery, and many chemical processes were expected to perform in a half-magical sort of way. The processes being promoted were considered in the nature of panaceas, capable of solving all the sewage problems of that day. When they failed to live up to expectations, prejudice against them naturally followed. Today the limitations of chemical treatment methods are better known. Those now interested in promoting them will be well advised if they take the attitude of making available an additional process to a well-developed art, rather than that of attempting to supplant existing methods by one that will solve all problems.

# NEWS OF THE WEEK

## Courts Disapprove Loan by R.F.C. to City of Ogden

OGDEN, Utah, has been prohibited by the state supreme court from accepting a loan of $745,000 from the Reconstruction Finance Corporation for additions and extensions of its waterworks system and improvements to the pumping equipment, as was noted briefly in these pages last week.

### Findings of the court

The court based its findings upon the conclusion that under the special circumstances surrounding the use to be made of the money the issue of water revenue bonds for sale to the R.F.C. would constitute an issue of bonds in excess of the legal limit for the city. Judge Folland, who rendered the decision of the court, stated that the court did not believe that the "special fund doctrine" put forward by the city authorities in support of their view that the bonds issue should not be considered as an increment to the debt subject to statutory limitations applied to this case because the contemplated improvement would result in no increase in revenue to the city water department. City officials maintained that the special fund doctrine did apply because the water department was operated as a distinct unit with its revenues segregated for the payment of interest and amortization, the excess being turned over to the city for general purposes. The court held, however, that the doctrine only would apply where the loan was used to finance new facilities that would produce revenues adequate for bond interest and amortization or for extensions that would bring in enough new business to carry the additional cost involved. As it was not alleged that the new improvements would materially increase the revenues of the entire system or that the income from them can be segregated or estimated the court said, "It follows necessarily, therefore, that the burden on the general taxpayer will be increased to make up to the general fund the amount which formerly went to the general fund, but now is to be impounded in the waterworks fund to meet the obligations of the water revenue bonds. Thus while tax revenues are not directly pledged to the payment of the water revenue bonds, the tax levy must be increased and such levy will be indirectly used to feed the special fund."

The court further cited the following quotation from the California Supreme Court, "We do not believe that the special fund doctrine was ever intended to be applied to a situation where the municipality, directly or indirectly, is or may be compelled to feed the special fund from other revenues and in addition to those arising from the special improvement contemplated. Such a subterfuge, if sanctioned, would go far to effectively wipe out the purpose and intent of the constitutional provision."

## Registration Fails in Washington

A bill providing for the registration of engineers and land surveyors in the state of Washington, introduced during the recent session of the state legislature, failed to pass, due to the opposition of large corporations.

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## R. F. C. Bids on Seattle Bonds

The Reconstruction Finance Corp. was the only bidder when the city of Seattle, Wash., offered $750,000 of city water bonds recently. The R.F.C. offered to take 5 per cent bonds at par. Its offer was in line with its agreement made several months ago to take $1,491,000 of water bonds.

On the recommendation of the board of public works, Seattle will construct a new $600,000 pipe line in the Cedar River watershed by contract. The use of day labor in replacing the existing pipe line has been considered by the city.

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## New York License Law Changes Affect Use of Term Engineer

Amendments to the engineer license law of New York State passed at the recent session of the legislature are designed to make it easier to obtain conviction for the misuse of the terms engineer and surveyor. As amended the law reads, "It shall be unlawful for any person to practice or to offer to practice professional engineering or land surveying in this state or to use the title engineer or surveyor or any other title, sign, card or device in such manner as to tend to convey the impression that such person is practicing professional engineering or land surveying or is a professional engineer or land surveyor, unless such person is duly licensed under the provisions of this article." Previously the law had the qualifying adjectives professional and land surveyor before the terms engineer and surveyor.

Another amendment relates to structures exempt from the provisions of the law. Previously the act exempted structures costing $10,000 or less. As amended the law exempts structures costing $10,000 or less and not exceeding 30,000 cu. ft. and alterations to any building or structure costing $10,000 or less which do not involve changes affecting the structural safety thereof. A similar provision has been included in the architect's law.

In both the architect's and engineer's law, under the section relating to examinations of candidates where the law heretofore permitted the examining board to exempt graduates from schools registered by the state department of education as having satisfactory courses in architecture and engineering, an exception has been included as follows, "except the written examination to establish his competency to plan and structurally design and supervise the construction of buildings and similar structures."

## Better Construction in Earthquake Zone Recommended by Jury

A RECOMMENDATION for the adoption of a county building code for the construction and repair of all buildings in Los Angeles County and the opinion "that every masonry building of any height, and every wood-frame building more than two stories high, be designed by either a licensed architect or a registered structural engineer and that every skeleton frame building be designed structurally by a registered structural engineer or a certified architect who qualifies as a structural engineer" were outstanding findings in the verdict of the coroner's jury following the eight-day inquest over the deaths resulting from the southern California earthquake of March 10. The jury was composed of construction experts and the inquest deliberately went beyond the mere determination of the cause of the deaths in order to find the reasons for the loss of life and to develop the possibility of providing more earthquake-resistant construction, particularly for schools and other public buildings.

The verdict of about 3,500 words summarized the general situation with respect to the recurrence of earthquakes, the deficiency in several types of present construction and concluded with specific recommendations for building requirements in the future. Much of the material in the verdict relating to faulty construction and to the relative amount of damage sustained by various types of buildings has already been covered by the *Engineering News-Record* reports in the issues of March 16 and 23; the more definite findings of the jury and some particularly pertinent quotations from the verdict follow:

### Character of the quake

In respect to the earth movement, the verdict found it to be of "moderately strong intensity," having an epicenter on the Inglewood fault in the ocean off Newport Beach with intensity greater along the fault line and gradually damping as the distance from the fault increased. Further, for equal distances from the fault and the same type of building, the greatest damage was done on soft, alluvial soil and the least to those buildings on harder foundations. The earthquake was not classed as a "great earthquake" and was not comparable to the San Francisco or Tokyo disasters but was considered comparable to the Santa Barbara earthquake of 1925. The concluding reference to the earthquake pointed out the possibility of future movements in the area but stated that there was no merit in the predicting of shocks and no evidence existed as to any future extreme earthquake.

### Effect on buildings

Referring to the effect of the earthquake and the lessons learned, the verdict pointed out that wood-frame buildings generally suffered only minor damage but lacked proper anchorage to foundations and had a particular hazard in the danger from falling chimneys. "Buildings of reinforced concrete and steel-frame buildings suffered very little other than incidental damage, most of which could have been avoided by proper design."

"Masonry buildings were the principal sufferers and their failure occasioned the principal loss of life. Damage was mostly confined to those built with poor quality of lime mortar, inadequate bonding and anchoring, or of inferior workmanship, and built to designs which took no account of horizontal forces." This conclusion in the verdict was further specifically directed to the wholesale damage sustained by school buildings in the area.

"Parapet walls were conclusively shown to be the weakest and most dangerous element of buildings as constructed and this experience points to the positive necessity of elimination or so designing and building them, that they will stay in place under earthquake conditions. . . . It was an outstanding fact that the greatest hazard from falling material was at the entrances of the buildings and along the sidewalks; and since it is an almost irresistible impulse with many people to run out of a building during an earthquake, it is a fact that the places where people were most likely to be or to go were the most dangerous."

"If it were found that criminal responsibility could be proven against any individual or individuals, it would no doubt serve as a salutary warning to others in the future; but a far more vital need is to bring the entire public to a realization of the fact that general laxity and indifference to inadequate building practices have existed in the past and to point out the importance of avoiding the continuance of such attitude as regards the future."

### Preparedness urged

The jury deprecated the idea of suppressing information on earthquake possibilities, favoring a program of reasonable preparedness and provisions for safety, which it concluded could be accomplished for a small additional cost.

"None of the ordinances in any portion of the damaged area adequately provide protection against horizontal or other stresses produced by earthquake oscillations. Stability of construction was often sacrificed for architectural effect or for purposes of utility or convenience, or with an eye to economy." Further, the jury found that unincorporated areas were not covered by building ordinances and in many cases structures were neither properly designed nor inspected.

### Specific recommendations

The verdict of the jury recommended that: 1. The uniform building code (developed by the Pacific Coast Building Officials Conference and now in use in scores of cities not only in California but a dozen other states) be immediately adopted as a legal code covering construction and, or repair of all buildings in Los Angeles County with an additional provision that a factor of 100 per cent against lateral forces be required in construction of all fire walls, towers, cornices, overhanging or projecting objects and with the exception that the use of lime mortar referred to in the code be eliminated.

2. Repairs to structures damaged but not justifying demolition should be guided by the principles and requirements of the Uniform Building Code.

3. That every masonry building of any height, and every wood-frame building more than two stories high, be designed by either a licensed architect or a registered structural engineer and that every skeleton-frame building be designed structurally by a registered structural engineer or certified architect who qualifies as a structural engineer, and that the construction of every such building be supervised by a registered structural engineer or a licensed architect who qualifies as a structural engineer.

4. State contractors' license law should be strengthened to require still greater ability on the part of contractors.

5. School buildings of more than a single story in height should be built of class A type of construction exclusively.

In conclusion the jury recommended that there be a consolidation of all school building affairs of the county in the office of a central organization.

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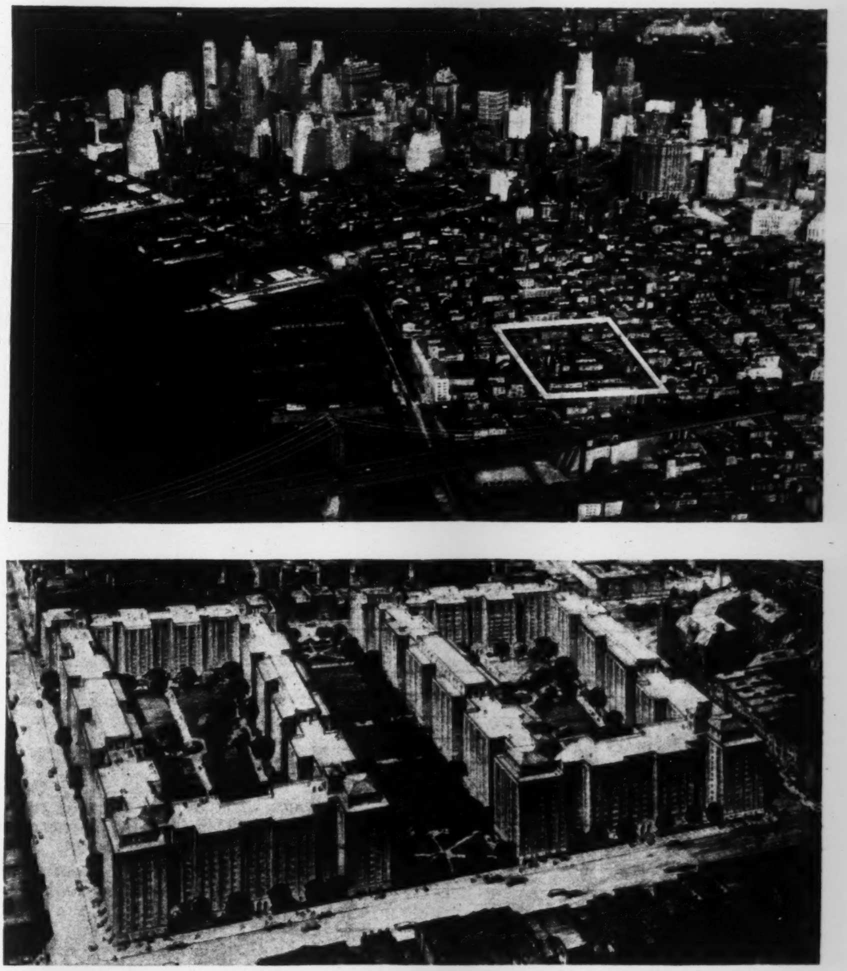
## Loan Made for Small Waterworks

The Reconstruction Finance Corp., on March 28, agreed to purchase $40,000 of waterworks debentures of the village of Hartland, Wis., the money to be used for the construction of a water supply system.



PROPOSED R.F.C.-FINANCED HOUSING IN NEW YORK CITY

The first units in Knickerbocker Village, the name given to the large-scale housing project on the lower east side of Manhattan Island, New York, for which the R.F.C. has agreed to loan $8,075,000 at 5 per cent, will cover a two block site containing 219,736 sq.ft. The site, located just south of the Manhattan Bridge, as shown in the upper view, is bounded by Catherine, Monroe, Market and Cherry streets and includes Hamilton St. which is to be vacated, if an agreement can be reached with the city. The site is now covered with some of the worst slum housing in the city. It is said that demolition work will start May 1, preparatory to erecting the two 12-story garden apartment units, which will provide 6,030 rooms in 1,630 apartments. Each unit is a hollow square, as shown in the lower view, enclosing a court 131 ft. wide and 230 ft. long. Between the units is a play ground 100x350 ft. The buildings cover 43.8 per cent of the site. Fireproofed steel frame construction is understood to be contemplated. According to Fred F. French, president of Fred F. French Operators, Inc., who obtained the loan "2,000 men will be engaged actually on the site, another 8,000 in shops, foundrys, factories, mines, lumber camps, etc." The Fred F. French Co. will act as general contractor and will receive two-thirds of its fee in income debentures of Knickerbocker Village, Inc. Fred F. French Operators, Inc., will manage the property when completed

## Contracts Let for Two Tunnels of Colorado River Aqueduct

Contracts for two more tunnels of the Colorado River aqueduct were let by the Metropolitan Water District of Southern California on April 7. Contract for the Valverde tunnel, 7.1 miles long, went to the Dravo Contracting Co., Pittsburgh, at $3,548,658, and contract for the 1.1 mile Bernasconi tunnel was awarded to Hamilton & Gleason Co., Denver, at $412,055. Under these contracts the tunnels are to be built to full size, 15 ft. 3 in. internal diameter. Contract for the 12.7-mile San Jacinto tunnel was let two weeks ago.

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## New York Law Amended in Favor of Engineers

The multiple dwelling law of the State of New York has been amended by a bill passed by both houses of the New York State legislature and now before the Governor for his approval which if approved will place engineers on a par with architects in the preparation and filing of plans for all structures covered by the law. Section 300 of the original law was so worded as to permit only owners and architects to file such plans. Attempts to amend the law to eliminate this discrimination, begun three years ago, have resulted in much controversy between the architectural and engineering professions in New York State. Last year the architects agreed to offer no further objections to the proposed amendment and this year an amendment was introduced into the legislature by Senator Thomas C. Desmond, an engineer, with the endorsement of both professions.

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## *Washington Notes*

By Paul Wooton
*Washington Correspondent*

Expeditious action on a public works program is forecast by President Roosevelt's determination to put more men to work. The administration has accepted the fact that any considerable volume of construction work must be financed by bond issues, although the President's plan will discard the self-liquidating principle that has restricted the operations of the Reconstruction Finance Corporation to date in financing water works and other community improvements.

Perplexed by the question of amortization, the administration has not yet committed itself to any definite scheme. It has been suggested for Mr. Roosevelt's consideration that financing would be facilitated by offering for popular subscription construction bonds of small denomination but there has been no intimation that this method will be adopted.

Senate leaders who have conferred with the President regard as impracticable the proposal to apply to the construction of useful works in local communities the federal-aid principle which has built most of the main highways out of 50-50 contributions by the federal and state governments. The belief is that municipalities do not have sufficient funds to assure any appreciable amount of construction on such a basis.

# Contractor Upheld in Louisiana Suit Over Creosoted Piles

THOUGH recognizing variation from creosoting specifications as causing subsequent failure of piling driven under sub-contract for P. Olivier & Son, Inc., general contractors for wharf unit No. 3 of the Lake Charles (La.) Harbor and Terminal District, the Louisiana supreme court, in a decision reversing the trial court, has upheld the contention of the contracting firm that inspectors' approval at the treating plant was final and precluded inspection in place.

The board of commissioners of the Lake Charles district, in 1931, received bids for the construction of the third addition to its wharf facilities, the contract being awarded to P. Olivier & Son, Inc., of Lake Charles, on a low bid of $444,000. Under the contract no sub-contractor was to be recognized and the creosoted material was to be given the full-cell treatment. The Robert W. Hunt Co., of Chicago, on June 23, 1931, was engaged to furnish inspection service on creosoted materials, treatment of which was to be effected by the American Creosote Works.

After 2,473 creosoted piling of a total of approximately 3,600 had been driven the drawing away of sand and clay from around a piling disclosed pile failures, and subsequent examination revealed that a majority of the piles showed signs of brooming and disintegration. Driving was suspended. The board (Sept. 24, 1931) demanded of the contractor that all piling in each tenth bent be drawn, each defective pile to be at contractor's expense and each good pile at the Board's expense. This the contractor refused to do. On Oct. 28, the board rejected all creosoted material on hand and in place, ordering its replacement, which also was refused. On Nov. 12 the board declared the contractor in default, taking over the work for completion. The contracting firm brought suit for payment under contract, and secured a court writ tying up funds pending disposition of the case.

### Inspection arrangements

In arranging for inspection with the Hunt company, Elmer E. Shutts, the board's engineer, asked for assignment of the company's most experienced men, to be continuously on duty, and to make all reports on material and treatment to him personally; he also instructed them to notify the board immediately should any irregularities in treatment occur or should its representatives have any difficulties.

The record shows, said the high court, that difficulty was encountered in getting some charges sufficiently dry, in accord with specifications, to satisfy the board's inspectors. The use of compressed air was brought up by the plant and subsequently applied with the knowledge and consent of the inspectors. A close record was kept by the plant showing the use of air, the quantities and duration of charge. This feature was not included in inspectors' reports to Mr. Shutts who claimed he was ignorant of the use of air until trial of the suit. Mr. Shutts was furnished with a certificate of inspection by Hunt's certifying that every stick of creosoted piling and lumber described therein and consigned to the building site had been inspected according to instruction and, based on their experience and judgment, were within the limits of the specification of Lake Charles Harbor and Terminal District furnished by them.

"There is little or no doubt," said the supreme court, "that the application of air on the material at a time when it was at its weakest and not physically able to stand the pressure, *i.e.*, after being subjected to steaming for several hours, punished it severely and caused it to eventually fail under the usual pounding of the hammer while being driven at the site."

In discussing the variance from specification, the court said in part: "It certainly was the duty of the inspecting laboratory to know what was or was not permitted under the specifications, details of which they were familiar with, and it certainly was their province to determine whether the process of treatment was or was not in accord with those specifications and to reject or accept the charges according to dictates of their own judgment. But the truth of the matter is that whilst the board looked for compliance with the specifications regarded in the light of process specifications, their inspectors required results as though the treatment was under what is known as result specifications, the different operations and their sequence are stated and little if any discretion left to the contractor, the owner accepting the responsibility for the results, whilst in result specifications what is wanted is stated but directions are not given as to how to obtain that result."

## Bids Concluded on Big Contracts for San Francisco Bay Bridge

With the receipt of bids on the San Francisco anchorage and the Yerba Buena Island crossing of the San Francisco-Oakland Bay bridge, opened on March 28 and 29, bids on all the major items of the bridge had been received by the California Toll Bridge Authority. The low bidder on the anchorage was the Healy Tibbitts Construction Co. of San Francisco, $1,036,500, and the low bidder on the Yerba Buena Island crossing was the Clinton Construction Co. of San Francisco, $1,821,292. Other bids and details of the bidding are given on p. 486 of this issue.

Other contracts for work on the bridge to be called for in the future are as follows:

| | |
|---|---|
| **(To be financed with R.F.C. funds)** | |
| Demolition, San Francisco section | $75,000 |
| San Francisco section | 950,000 |
| Substructure, girder spans on mole | 200,000 |
| Move buildings on Yerba Buena Island | 50,000 |
| Lightweight concrete aggregate | 80,000 |
| Administration and toll buildings | 350,000 |
| Electrical work | 350,000 |
| Wrapping cables | 250,000 |
| Operating equipment | 350,000 |
| Final field painting | 500,000 |
| Total | 3,155,000 |
| (To be Financed With State Highway Funds) | |
| Electrical work | $100,000 |
| Demolition, San Francisco approach | 25,000 |
| San Francisco approach | 675,000 |
| Mole construction and waterfront grading | 1,200,000 |
| East-Bay viaducts | 1,000,000 |
| Southern Pacific underpass | 160,000 |
| Paving East-Bay streets | 780,000 |
| Sub-Total | 3,940,000 |
| Grand Total | 7,095,000 |

## Housing Bill Approved in Jersey

A bill providing for the incorporation of public housing corporations has been approved by the governor of New Jersey. The bill authorizes the setting up of limited-dividend corporations for the construction of low-cost housing under regulations set up by the Public Utilities Commission. It is similar to the New York housing law, the first of such laws to be enacted in this country.

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## Bids Called for Power Work

Contractors' bids will be received up until April 26 by the city of Seattle, Wash., for the completion of the Diablo power house, to be paid for in city light bonds, and for furnishing $1,854,000 in cash through the purchase of light bonds to retire outstanding warrants of the city light department used in the purchase of equipment for the power plant. Completion of the plant will enable the city to about double its present output.

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## Control of Water Resources Planned in State of Maryland

A bill providing for the creation of an unpaid commission of five members to regulate and supervise the use of the state's water resources has been approved by both houses of the legislature of Maryland. The commission will consist of the chief sanitary engineer of the state department of health, the chairman of the public service commission, the state geologist and two others to be appointed by the Governor. The commission will have among its functions the supervision of dams and reservoirs. Persons, corporations or political subdivisions of the state must apply to the commission in writing before appropriating any waters, either surface or underground.

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## Engineer Chosen to Direct Federal Building Construction

L. W. Robert, Jr., consulting engineer and architect of Atlanta, Ga., has been nominated Assistant Secretary of the Treasury to succeed F. K. Heath, who resigned in March but has continued in office until his successor was nominated. As Assistant Secretary of the Treasury, Mr. Robert will have charge of the Office of the Supervising Architect which handles plans and contracts for all federal buildings, also the general supply committee which serves as the purchasing agent for the ordinary supplies for government departments and agencies.

Mr. Robert is a native of Monticello, Ga., and graduated from the civil engineering department of the Georgia School of Technology in 1908. In 1909 he received his electrical engineering degree from the same institution. For several years before graduation he was on railroad construction work and since graduation has specialized in industrial building work, first with Park A. Dallis, mill architect and engineers at Atlanta. In 1917 he organized firm Dallis-Robert Co., architects and engineer,s at Atlanta. In 1917 he organized the firm of Robert & Co., industrial architects and mill engineers with headquarters at Atlanta.

## SOCIETY CALENDAR

**AMERICAN SOCIETY OF CIVIL ENGINEERS**, annual convention, Chicago, June 27-30.

**AMERICAN SOCIETY FOR TESTING MATERIALS**, annual meeting, Chicago, June 26-30.

**AMERICAN WATER WORKS ASSOCIATION**, annual meeting, Chicago, June 12-16.

**AMERICAN WELDING SOCIETY**, annual meeting, New York, N. Y., April 27-28.

**NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS**, annual convention, Chicago, June 26-28, 1933.

**SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION**, annual meeting, Chicago, June 27-30.

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**ENGINEERS WEEK**, Century of Progress Exposition, Chicago, June 25-30.

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**DETROIT ENGINEERING SOCIETY** will hold its annual meeting on May 4. C. F. Kettering will be the principal speaker.

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## *Brief News*

REPAIRS OF THE DIKE around the Luken airport at Cincinnati, Ohio, which was damaged during the recent flood, are being made by the city council. The dike is to be widened, raised 2 ft. and reinforced by the planting of trees.

CONSTRUCTION OF WATERWORKS, sewer and gas systems as well as the financing and operating of the system by a county, city or incorporated town in Alabama is now permitted under a bill recently approved by the Governor of the state.

THE PORTLAND CEMENT ASSOCIATION has reopened its St. Louis office after a shutdown of two years. A. C. Frech, district engineer, will have charge of the office. The reopening is due to an improvement in the prospects for the cement industry in the St. Louis region.

LACK OF EVIDENCE that illuminating gas was present in the sewer system of Ottawa, Ont., in sufficient amount to cause the destructive explosion that occurred on Jan. 28, 1931, has resulted in the dismissal of a suit brought by the city against the Ottawa Gas Co. to collect $376,364 damages.

A NEW PUMPING PLANT is to be built by the city of Augusta, Ky., as a result of having its old plant flooded out by the high water in the Ohio River valley. The city has made a tentative agreement for the installation of a new plant above the 1913 flood line and a new 100-ft. pumping well.

EXTENSION OF GRAND CENTRAL PARKWAY, Long Island, to Flushing Bay, and extension of Interborough Parkway to Grand Central Parkway and to Highland Boulevard, also the extension of Southern State Parkway to Sunrise Highway, and the completion of construction of the Grand Central Parkway from the Nassau County line to Queens Boulevard at a total cost of $3,000,000 is provided for in a bill recently approved by the Governor of New York State. Of the money appropriated, $19,400 is to be made available to the Long Island State Park Commission for engineering services.

THE FIRST OF EIGHTEEN CONCRETE CAISSONS for the Santa Monica, Calif., yacht harbor breakwater has been towed to Santa Monica from Long Beach, Calif., where construction of the other units by the Puget Sound Bridge and Dredging Co., and W. F. Way, Inc., is in progress. The caisson, weighing 2,100 tons, is 111 ft. long, 35 ft. wide and 36 ft. deep. The towing distance from Long Beach to Santa Monica is 30 miles. The caisson is made up of three elliptical cylinders bound together vertically. It was poured in a week and under the contemplated schedule the remaining seventeen will be produced at the rate of one each week.

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## *Personal Notes*

ADOLPH J. ACKERMAN, until recently chief engineer for the contractors on Madden Dam in the Canal Zone, has been appointed an hydraulic engineer on the engineering staff of the Canal Zone.

R. G. CONE, whose appointment to be resident engineer for the Strauss Engineering Corp. on the Golden Gate bridge was noted in our issue of March 30, was engineer on the Ambassador bridge at Detroit, not on the "bascule" bridge, as stated in that notice.

JOHN H. CARRUTH, major, Corps of Engineers, who has been on duty as a student at the Army War College in Washington, has been appointed assistant to the division engineer, Gulf of Mexico division, and ordered to New Orleans as soon as his duties at the War College are completed.

EDWARD E. SPENCE, engineer and pioneer resident of Beverly Hills, Calif., has been appointed mayor of that city to succeed Paul E. Schwab, who resigned recently. The appointment was made by the city council, of which Mr. Spence has been a member since 1923. Mr. Spence was an engineer for the Rodeo Land and Water Co., original subdividers of the city.

JAMES W. CAREY, of Seattle, has been named chief engineer of the department of public works of the State of Washington succeeding Harper Craddock who will remain in the department as assistant to Mr. Carey. Mr. Carey formerly was chief engineer of the department but resigned in 1925. He is a member of the firm of Carey & Harlin, rate and valuation specialists but will resign from the firm on assuming his new duties.

SIDNEY H. BIRDSEYE, former employe of the U. S. Geological Survey and recently associated with Brock & Weymouth as ground engineer, has been appointed chief of the demarcation commission to identify, survey and mark the boundary between Guatemala and Honduras in accordance with the award made January 23 by the special tribunal of which Chief Justice Charles E. Hughes was president. Mr. Birdseye will sail from New Orleans on April 12 to join the Guatemalan and Honduran engineers who will work with him on the survey.

L. N. GREEN, former structural designer and construction engineer for Brown Co., Berlin, N. H., and designing draftsman for Parker-Young Co.; ARTHUR SNODGRASS, former field engineer for Brown Co.; C. L. THOMPSON, construction engineer for the same company; HARLAN JORDAN, architect, and HAYDEN FANCY, field engineer for the New England Power Co., have established a consulting engineering office at Berlin, N. H., for general engineering work including pulp and paper operation under the name of Engineering Services.

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## Obituary

**William P. Dallett**, president of the W. P. Dallett Co. of Philadelphia, died there on March 31.

**Conrad P. Darling**, engineer of Huntington, Long Island, died there on April 7 from injuries received in an automobile accident.

**John P. Randerson**, for many years a dredging contractor on federal work on the Atlantic seaboard, died at his home in Albany, N. Y., on April 3, age 77 years.

**Walter H. Wendler**, Southwestern representative of the Leadite Co., died on March 27, age 30 years. Mr. Wendler was a graduate of the A. & M. College and since graduation has been active in sanitary engineering work in the Southwest.

**Dennis A. Blakeslee**, president of the Blakeslee-Rollins Corp. of Boston, died at his home in New Haven, Conn. on April 5, age 76 years. Mr. Blakeslee has been active in construction work for many years both as head of the Blakeslee-Rollins Corp. and as head of the firm of C. W. Blakeslee & Sons, New Haven, a firm founded by his father.

**Lee H. Miller**, chief engineer of the American Institute of Steel Construction, died at Cleveland, Ohio on April 9 following several weeks illness. Mr. Miller was born in Aylmer, Ontario, in 1877 and graduated from the school of Applied Science and Engineering of Toronto University in 1900. Following graduation he came to this country and worked with various manufacturers of engineering equipment until 1908 when he became structural sales agent for the Bethlehem Steel Co. at Cleveland. In 1922 Mr. Miller started the work of organizing the American Institute of Steel Construction and became its first managing director and subsequently chief engineer.

**Philip H. Bevier**, chief engineer of the National Fireproofing Corp., died at his home in Brooklyn, N. Y. on April 5, age 74 years. Mr. Bevier graduated from the civil engineering department of Rutgers University in 1882 and for several years after graduation was in the New Jersey geographical survey and on the Nicaragua Canal survey. Thirty-five years ago he became an engineer for the National Fireproofing Corp.

## Contracts and Capital

### CONSTRUCTION

(Thousands of Dollars)

| | Weekly Average Apr., 1932 | Four Weeks | Week Apr. 13, 1933 |
|---|---|---|---|
| Federal government | $2,912 | $2,500 | $2,958 |
| State and municipal | 14,799 | 5,371 | 6,858 |
| Total public | $17,686 | $7,871 | $9,816 |
| Total private | 7,882 | 5,048 | 3,230 |
| Week's total | $25,586 | $12,919 | $13,046 |
| Cumulative to date | | | |
| 1932 | | $320,647 | |
| 1933 | | 244,141 | |

### NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

| | Weekly Average Apr., 1932 | Four Weeks | Week, Apr. 8, 1933 |
|---|---|---|---|
| New capital issues | $11,500 | $1,643 | $1,981 |
| Cumulative to date: | | | |
| 1932 | | $260,500 | |
| 1933 | | 70,867 | |

| R.F.C. self-liquidating loans: | |
|---|---|
| Approved current week | $40,000 |
| Approved to date (including 1932) | 202,171 |
| Actual loans made (including 1932) | 20,461 |

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## *ENR* Cost and Volume Index

| **E.N.-R. Cost** | | **E.N.-R. Volume** | |
|---|---|---|---|
| **April, 1933** | **160.16** | **March, 1933** | 62 |
| March, 1933 | 158.44 | February, 1933 | 82 |
| April, 1932 | 153.12 | March, 1932 | 97 |
| 1932 (Average) | 156.97 | 1932 (Average) | 127 |
| 1931 (Average) | 181.35 | 1931 (Average) | 220 |
| 1930 (Average) | 202.35 | 1930 (Average) | 260 |

1913 Average........ 100

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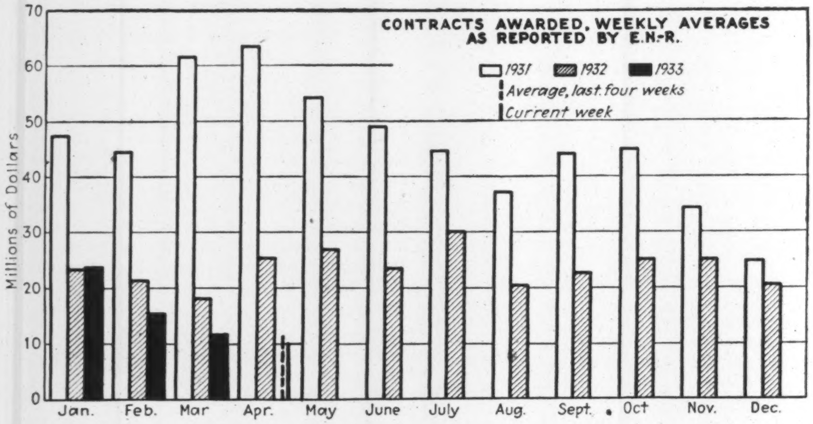
## Engineering Contracts and Capital

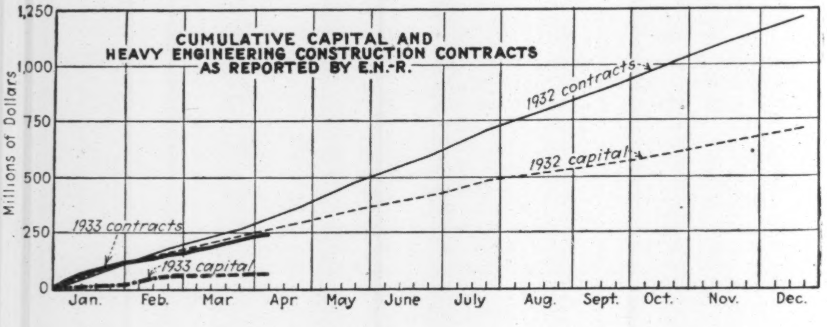
Award of two additional tunnel contracts on the Colorado River aqueduct raised the total of heavy engineering construction contracts reported the past week above the average of the past four weeks. The past week's total is $13,046,000, below the previous week's total of $17,256,000, but slightly above the four-week average of $12,919,000. Public works contracts amounted to $9,816,000, the largest in ten weeks. Federal awards remained low with a total of $2,958,000. Private contracts dropped back to the unusually low total of $3,230,000 following the previous week's high mark of over 10 millions, swelled by a single housing award. Highway lettings were the lowest in two months, aggregating only $919,000. Highway work, as well as most all public work, is being held up until a definite federal program is announced. No contracts as yet have been awarded for the San Francisco-Oakland bridge, though bids have been opened for most major sections. Cumulative contracts let to date this year total $244,141,000, as compared with $320,647,000 for a similar period last year. Two large contracts let during the past week were the Valverde tunnel on the Colorado River aqueduct, and the St. Louis court house.

Several large proposed projects were announced last week, including additional work on the San Francisco-Oakland bridge to cost $7,000,000; highway work in Missouri aggregating 1.5 millions, bids to be opened next week; a new union station at Los Angeles to cost $2,000,000, postponement of which has been denied by the I.C.C.; harbor improvement work at Los Angeles to cost over 3 millions; a brewery at Manville, N. J., estimated at $1,000,000; a drainage canal and reservoirs in Arkansas, $10,000,000; a bridge near Yorktown, Va., estimated at $1,200,000; and an oil refinery at Houston, Tex., $1,000,000.

New productive capital issues sold last week total $1,981,000, all public bonds. Of this amount, $779,000 represents purchases by the Reconstruction Finance Corp. on previous commitments. New productive capital issues sold to date this year total $70,867,000, as compared with a total of $260,500,000 for the same period last year.

The accompanying chart was made prior to receipt of notice of award of the two Colorado aqueduct tunnels totaling $3,961,000, and the curve of the current week's total is therefore in error. This curve should read $13,046,000, instead of less than $10,000,000 as shown.

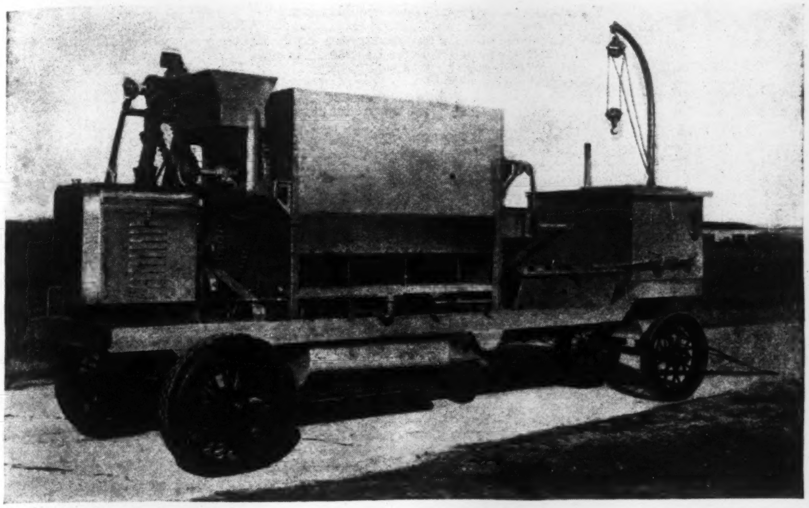




# *Construction Equipment and Materials*

## Highway Maintenance Machine

A new street and highway maintenance machine is announced by the White Mfg. Co., Elkhart, Ind., for bituminous road and street repair work or for resurfacing brick and concrete. The machine consists of a stone and sand dryer, pug mixer, detachable bucket elevator, tar kettle, oil burners, measuring devices, all mounted on rubber-tired wheels. It comprises a complete small asphalt plant to produce high temperature mixes for immediate hot application or cooler mixes for deferred or cold laying.



**Machine comprises dryer, pug mixer, bucket elevator, tar kettle, oil burners and measuring devices.**

The machine follows the general design of the Chausse-White street repair trucks. The new design, however, makes it applicable for production of cold laying mixtures by having the pug mixer in the open and away from fire. Its capacity is rated at 25 tons per day of high temperature mix and 40 tons per day of low or medium temperature mix.

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## Removable Drill Bit

The Timken Roller Bearing Co., Canton, O., has recently put on the market a new type of removable drilling bit which it is claimed eliminates the difficulties heretofore encountered in attempts to develop such a bit. Replacing the conventional bit forged on the end of a hollow drill steel, the new bit is held tightly against an upset shoulder on the steel by means of a special left-hand thread designed for strength and easy removability. The left-hand thread is opposed to the direction of rotation of the steel so that the bit is kept tightly against the shoulder while drilling. The hammer blow is transmitted from the steel through the shoulder to the body of the bit, none of its force being absorbed by the thread.

**New type of removable drill bit made by Timken Roller Bearing Co.**

When the bit becomes too dull for further service it can be quickly removed and replaced by a new one. The manufacturer states that experience on many operations has already shown it to be more economical to throw the used bit away than to resharpen the conventional type drill.

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## *New Equipment in Brief*

*Light Hoist.* The Newo Hoist Co., 17309 Fernway Road, Cleveland, Ohio, has developed a fractional-ton hand hoist for light service with several unusual features. A positive locking device holds the load in any position. The hoist cord is of a special non-skid rubberized fabric. The ratchet sheaves turn only when the load is being raised and lock automatically when the load is being lowered.

*Geared Motors.* The U. S. Electrical Manufacturing Co., Los Angeles, Calif., announces the U. S. Syncrogear motor, which is a self-contained, single-base, geared transmission, supported on a heavy gear pedestal base designed to withstand the torsional stresses inherent in geared, high-torque reduction. Motor and reduction gears are contained within an integral housing. The unit is furnished for speeds from 2 to 10,000 r.p.m.

*Power Shovels.* Harnischfeger Corp., Milwaukee, Wis., has developed a new line of excavators incorporating what the company calls "split-second control." The company states that a new sensitive power clutch enables the operator to feel immediately the resistance to the dipper. A small trigger attached to the hoist lever trips the bucket. Other features are incorporated in the design of the machine which are designed to speed up operation.

## *New Publications*

***Wood Preservers.*** C-A-Wood Preserver Co., St. Louis, Mo., has published a 14-page mimeographed pamphlet entitled "The Evolution of Brush and Spray Wood Preservers Specifications."'

*Sheet Copper.* The American Brass Co., Waterbury, Conn., has issued a 10-page bulletin describing Anaconaa "Electro-Sheet" for built-up asphalt rooms. "Electro-Sheet" is thin sheet copper weighing 2 ounces per square foot.

*Electric Clocks.* Warren Telechron Co., Ashland, Mass., has published bulletin A-12 which describes and illustrates the company's line of commercial clocks, program clocks, program signaling equipment, tower clocks and central control systems.

*Wire Rope.* A 27-page catalog has been published by the Macwhyte Co., Kenosha, Wis., describing single and multiple part wire rope slings. The slings described are of the Atlas, Drew, and Monarch types, manufactured by the Macwhyte Co.

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## *Business Notes*

THE SCOVILL MANUFACTURING CO. of Waterbury, Conn., has been licensed by the Dardelet Theadlock Corp. to manufacture and sell "Rivet-Bolts" and other bolts and nuts with the Dardelet self-locking thread.

THE NATIONAL WOOD PIPE CO., a new organization, has acquired the property formerly occupied by the Standard Wood Pipe Co. at Williamsport, Pa., and will manufacture and sell a full line of wood pipe.

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## New Concrete Vibrator of Internal Type

The demand for an effective mechanical means for placement of dry concrete has resulted in the development of the vibrating blade which has been added to the line of air driven vibrators sold by Munsell Concrete Vibrator, 999 West Side Ave., Jersey City, N. J. The vibrating blade consists of a half oval steel bar ½x2 in. in section by 3 to 15 ft. long (3 ft. long is standard) which is rigidly attached to the vibrator body. In operation the blade is placed in the concrete as deposited in the forms and the vibration permits the escape of the entrained air and water as the concrete is compacted. As the blade can be operated close to the forms, it eliminates the collection of air and water bubbles on the forms and results in smooth surfaces. The blade is recommended for construction work in columns, thin walls, deep girders and beams; in concrete products, for cast pipe, burial vaults, concrete piling, ornamental work, etc. It is especially recommended in any construction work where the reinforcement steel is closely spaced. Model type "C" is portable, and weighs 35 lb. Type "A", also portable, and weighing 35 lb., is equipped with a pronged spud for vibrating flat slabs, shallow beams, or girders. Type "B" which weighs 45 lb., is equipped with a vise clamp in order to allow rigid attachment to walers or studs.

**Munsell vibrator.**

Further information and engineering data can be secured by addressing Munsell Concrete Vibrators, 999 West Side Avenue, Jersey City, New Jersey.

# Unit Prices—Connecticut River Bridge, Hartford

BIDS WERE OPENED March 13 by the state highway department of Connecticut for the construction of a new suspension bridge over the Connecticut River at Hartford. No contracts will be awarded until appropriations are made by the state legislature. The bridge will be approximately 7,500 ft. long. The main span is a cable strand suspension bridge, 856 ft. center span and two 410-ft. suspended side spans. Cables will be made up of 1⅝-in. strands, compacted into a hexagonal cable about 15 in. in diameter. Two cables will carry a 40-ft. concrete roadway and two 12½-ft. sidewalks. Approaches are mostly deck plate girder spans. Foundations for the river piers will go down 100 ft. Pipe pile and spread footing foundations will carry the approaches. Robinson & Steinman of New York are consulting engineers and prepared the designs.

Substructure bids ranged from $2,364,826 to $3,070,451, eight firms submitting bids. The substructure unit prices given herewith are those of the three low bidders as follows: *A*—Poirier & McLane, New York, $2,364,826; *B*—Brader Const. Co., New York, $2,469,988; *C*—Dravo Contracting Co., Pittsburgh, $2,588,939. Only one bid was submitted for the superstructure, that of John A. Roebling's Sons, Inc., Trenton, for $2,056,400. Both sets of unit prices follow:

SCHEDULE OF UNIT PRICES — SUBSTRUCTURE

| ITEMS | Quant. | Unit Prices | | |
|---|---|---|---|---|
| | | A | B | C |
| Unclassified Excavation — Above El. + 5.0 | | | | |
| Main piers, c.y. | 900 | $3.50 | $1.00 | $1.70 |
| Cable bent piers, c.y. | 1,030 | 3.55 | 1.00 | 1.40 |
| Anchorages, c.y. | 4,000 | 3.50 | 1.00 | .80 |
| Walls and abutments in Hartford, c.y. | 1,800 | 3.45 | 1.00 | 1.70 |
| Silver Lane walls and abutments, c.y. | 1,200 | 3.40 | 1.00 | 1.40 |
| Piers W1 to W14 inclusive, c.y. | 590 | 5.40 | 1.00 | 7.00 |
| Piers W15 to W27 and N1 to N13 incl., c.y. | 3,130 | 5.85 | 2.00 | 4.00 |
| Piers E1 to E45 inclusive, c.y. | 4,820 | 4.50 | 2.00 | 1.00 |
| Unclassified Excavation — Below El. + 5.0 | | | | |
| Main piers, c.y. | 7,510 | 4.43 | 5.00 | 4.00 |
| Cable bent piers, c.y. | 3,550 | 4.63 | 5.00 | 3.00 |
| Anchorages, c.y. | 11,970 | 4.13 | 5.00 | 1.50 |
| All viaduct piers, c.y. | 4,620 | 6.87 | 5.00 | 5.00 |
| Unclassified Excavation — Pneumatic | | | | |
| Main piers, c.y. | 1,990 | 13.45 | 10.00 | 8.00 |
| Cable bent piers, c.y. | 2,540 | 15.96 | 10.00 | 7.00 |
| Anchorages, c.y. | 6,490 | 14.44 | 10.00 | 7.00 |
| Rock Excavation — Pneumatic | | | | |
| Main piers, c.y. | 300 | 29.22 | 10.00 | 20.00 |
| Cable bent piers, c.y. | 260 | 18.50 | 10.00 | 16.00 |
| Anchorages, c.y. | 1,150 | 18.50 | 10.00 | 16.00 |
| Caisson Concrete | | | | |
| Main piers, c.y. | 3,740 | 7.67 | 10.00 | 10.00 |
| Cable bent piers, c.y. | 4,010 | 7.67 | 10.00 | 10.00 |
| Anchorages, c.y. | 15,400 | 7.67 | 10.00 | 10.00 |
| Caisson Seal Concrete (Pneumatic) | | | | |
| Main piers, c.y. | 835 | 7.67 | 10.00 | 8.00 |
| Cable bent piers, c.y. | 865 | 7.67 | 10.00 | 8.00 |
| Anchorages, c.y. | 3,330 | 7.67 | 10.00 | 8.00 |
| Caisson Plug Concrete | | | | |
| Main piers, c.y. | 765 | 7.67 | 10.00 | 7.00 |
| Cable bent piers, c.y. | 690 | 7.67 | 10.00 | 7.00 |
| Anchorages, c.y. | 1,860 | 7.67 | 10.00 | 8.00 |
| Concrete Above Caissons | | | | |
| Main piers, c.y. | 6,700 | 7.67 | 10.00 | 9.00 |
| Cable bent piers, c.y. | 4,880 | 7.67 | 10.00 | 9.00 |
| Anchorages, c.y. | 16,910 | 7.67 | 10.00 | 9.00 |
| Concrete for Viaduct Piers, Walls and Abut's. | | | | |
| All walls and abutments except at E38, c.y. | 3,480 | 7.67 | 10.00 | 12.00 |
| Piers W1 to W14 incl., including 32 stair piers, c.y. | 490 | 7.67 | 10.00 | 10.00 |
| Piers W15 to W27 and N1 to N13 incl. except pier W24, c.y. | 1,690 | 7.67 | 10.00 | 9.00 |
| Pier W24, c.y. | 1,480 | 7.67 | 10.00 | 8.00 |
| Piers E1 to E23, E26 to E28, E30, E34 to E45, c.y. | 7,810 | 7.67 | 10.00 | 12.00 |
| Piers E24, E25, E29, E31, E32, E33, c.y. | 3,140 | 7.67 | 10.00 | 10.00 |
| Granite Facing | | | | |
| Main piers, c.y. | 1,105 | 65.00 | 70.00 | 74.00 |
| Cable bent piers below sidewalk, c.y. | 1,655 | 65.50 | 70.00 | 74.00 |
| Anchorages, below sidewalk, c.y. | 2,070 | 71.20 | 70.00 | 74.00 |
| On steelwork between anchorages and cable bents including conc. backing, c.y. | 520 | 103.40 | 80.00 | 90.00 |
| Piers W22 and W24, C. Y. | 165 | 67.50 | 80.00 | 100.00 |
| Ramp walls and abutments, c.y. | 400 | 93.80 | 80.00 | 145.00 |
| Concrete-Filled Steel Piles — 20 In. O.D., ½ In. Walls in Place Including Exc. and Conc. Fill | | | | |
| In Hartford, lin.ft. | 18.490 | 4.30 | 4.20 | 5.30 |
| In E. Hartford — E1 to E21 incl., lin.ft. | 24,090 | 4.30 | 4.20 | 5.50 |
| In E. Hartford — all others, lin.ft. | 80,570 | 4.30 | 4.29 | 5.50 |
| Structural Steel | | | | |
| Anchor chains and supports, lb. | 270,000 | .064 | .10 | .08 |
| Beams, cols., etc., between ramp walls, lb. | 459,000 | .03 | .05 | .04 |
| Anchor bolts, grillages and beams in foundations, lb. | 310,000 | .042 | .03 | .06 |
| Main pier caissons, lb. | 845,000 | .041 | .05 | .05 |
| Anchorage and cable bent caissons, lb. | 327,000 | .04 | .05 | .05 |
| Reinforcing Steel | | | | |
| In roadway, curbs and sidewalks on substructure steelwork, lb. | 145,000 | .035 | .04 | .04 |
| In piers E1 to E45 incl., lb. | 275,000 | .08 | .04 | .04 |
| In main piers, cable bent piers and anchorages, lb. | 1,476,000 | $0.035 | $0.04 | $0.04 |
| In all other piers, walls and abutments, lb. | 347 000 | .035 | .04 | .04 |
| Miscellaneous | | | | |
| Clearing and grubbing, total | ..... | 2,100 | 1.00 | 2,000 |
| Moving buildings — In Hartford, total | ..... | 15,000 | 20,000 | 17,000 |
| Moving buildings—In East Hartford, total | ..... | 7,500 | 10,000 | 6,000 |
| Wrecking buildings—In Hartford, total | ..... | 3,500 | 1,000 | 1,000 |
| Wrecking buildings—In East Hartford, total | ..... | 1,500 | 1,000 | 600 |
| Landscaping—In Hartford, total | ..... | 11,415 | 500 | 4,000 |
| Landscaping—In East Hartford, total | ..... | 864 | 300 | 1,200 |
| Roadway exc.—Unclassified—In Htfd., c.y. | 11,100 | 1.00 | 1.00 | 1.50 |
| Roadway exc.—Unclassified—In E. Htfd., c.y. | 7,200 | 1.00 | 1.00 | .80 |
| Fill at approaches and east anchorage, c.y. | 19,000 | .10 | 1.00 | .50 |
| Concrete roadway, curbs and sidewalk on substructure steelwork, c.y. | 755 | 12.40 | 10.00 | 20.00 |
| Concrete roadway off structure incl. gravel base, sq.yd. | 17,800 | 1.73 | 2.00 | 3.00 |
| Concrete sidewalks off structure including cinder base, sq.ft. | 38,800 | .18 | .25 | .30 |
| Concrete curbs off structure including cinder base, lin.ft. | 6,850 | 1.06 | 1.00 | 1.20 |
| Additional cost—White double curbs, lin.ft. | 2,100 | .32 | 2.00 | 1.00 |
| Additional cost—White single curbs, lin.ft. | 6,850 | .18 | 1.00 | .50 |
| Additional cost—White lane markers, lin.ft. | 2,700 | .27 | .50 | .20 |
| Asphalt roadway paving incl. concrete base and subgrade, sq.yd. | 4,450 | 3.03 | 2.50 | 3.50 |
| Granite curbs including cinder base, lin.ft. | 4,650 | 1.54 | 2.50 | 2.50 |
| Ornamental pylons at west end complete except lighting, total | ..... | 10,033 | 20,000 | 10,000 |
| Ornamental granite work above sidewalk at anchorages, total | ..... | 31,250 | 18,000 | 40,000 |
| Ornamental granite work above s. walk at cable bent piers, total | ..... | 8,500 | 15,000 | 5,000 |
| Metal hand rail on ramp walls and abutment at E38, lin.ft. | 1,690 | 4.50 | 5.00 | 6.00 |
| 24-in. woven wire fabric fence-timber posts, lin.ft. | 485 | 5.20 | 1.00 | 1.00 |
| Inertol waterproofing—Piers, walls, abutments and W. anchorage, sq.ft. | 24,500 | .054 | .05 | .05 |
| Malleable iron drainage scuppers on ramps, lb. | 2,000 | .125 | .05 | .15 |
| Street drainage catch basins—new and relocated, each | 61 | 79.50 | 40.00 | 120.00 |
| 8-in. tile drains, lin.ft. | 710 | 1.69 | .60 | .50 |
| 10-in. tile drains, lin.ft. | 1,700 | 2.57 | 1.00 | 1.00 |
| 24-in. tile drains, lin.ft. | 410 | 5.48 | 2.00 | 3.00 |
| 5-in. W. I. pipe, lin.ft. | 330 | 4.00 | 4.00 | 2.50 |
| 18-in. extra heavy C. I. pipe, lin.ft. | 120 | 6.00 | 4.00 | 7.00 |
| Steel doors in abutments and w. anchorage, | 4 | 150.00 | 30.00 | 120.00 |
| Ventilating grilles including bronze screens in abutments, each | 26 | 20.00 | 30.00 | 25.00 |
| Hockanum river diversion—excavation, c.y. | 6,000 | .52 | .10 | .50 |
| Hockanum river diversion—Fill, c.y. | 20,000 | .10 | .20 | .30 |

SCHEDULE OF UNIT PRICES — SUPERSTRUCTURE

| ITEMS | Quantities | | Unit Price |
|---|---|---|---|
| Structural steel — main towers | 2,400,000 | lbs. | $0.0634 |
| Structural steel — cable bents | 138,000 | lbs. | .0655 |
| Structural steel — stiff. truss and floor system | 3,635,000 | lbs. | .052 |
| Structural steel — truss and fl. sys. betw. anchorages and c. bents | 1,690,000 | lbs. | .0457 |
| Struct. steel — abutment W0 to bent W21 including ½ of bent W21 | 3,350,000 | lbs. | .0418 |
| Struct. steel — abutment N0 to bent N11 including ½ of bent N11 | 1,150,000 | lbs. | .0489 |
| Struct. steel — bents W21 and N11 to bent W24 including bent W24 | 2,020,000 | lbs. | .0485 |
| Struct. steel — bent W24 to w. anchorage | 4,170,000 | lbs. | .0462 |
| Struct. steel — E. anchorage to bent E38 | 9,723,000 | lbs. | .0359 |
| Struct. steel — bent E38 to abut. E46 | 846,000 | lbs. | .0381 |
| Strand plates, shims and bolts | 98,800 | lbs. | .0628 |
| Supports for hand rope and beacons | 13,500 | lbs. | .196 |
| Splay hoods | 26,000 | lbs. | .0663 |
| Stairways complete except foundations, lump sum | .......... | | 7,049 |
| C. S. tower and cable bent saddles | 96,000 | lbs. | .10 |
| C. S. cable bands and splay castings | 97,800 | lbs. | .152 |
| C. S. strand bearing blocks | 28,000 | lbs. | .122 |
| C. S. strand, suspender and hand rope sockets | 33,000 | lbs. | .20 |
| C. S. bearings for viaduct and susp. spans | 225,000 | lbs. | .0685 |
| C. S. curb expansion gratings | 2,200 | lbs. | .28 |
| Malleable iron drainage scuppers — C. bent to C. bent and type "B" | 33,000 | lbs. | .109 |
| Malleable iron drainage scuppers — type "A" | 16,000 | lbs. | .0943 |
| Malleable iron drainage scuppers — types "D" and "F" | 7,000 | lbs. | .0851 |
| Cable strand — 1⅝ in. dia | 1,306,000 | lbs. | .13 |
| Wrapping wire — No. 9 | 66,100 | lbs. | .227 |
| Suspender rope — 1⅛ in. dia | 8,230 | l.f. | 1.35 |
| Hand rope — ¾ in. dia | 6,590 | l.f. | .191 |
| Aluminum cable fillers C. bent to C. bent (per cable) | 3,065 | l.f. | 4.87 |
| Alum. cable fillers C. bent to anchorage (per cable) | 420 | l.f. | 7.37 |
| Zinc cable collars at saddles | 16 | each | 43.75 |
| C.I. cable sleeves at sidewalk | 4 | each | 113.33 |
| High tension bolts | 7,400 | lbs. | .11 |
| Bronze bearings | 2,870 | lbs. | .20 |
| Concrete in roadway, curbs and sidewalks on suspended spans and viaducts | 11,600 | c.y. | 16.00 |
| Reinforcing steel — roadway, curbs and sidewalks | 3,260,000 | lbs. | .0313 |
| 42 in. hand rail — anchorage to anchorage | 3,780 | l.f. | 4.94 |
| 42 in. hand rail — viaducts | 10,500 | l.f. | 4.71 |
| 24½ in. hand rail — viaducts | 6,620 | l.f. | 3.92 |
| 5 in. W. I. drain pipe | 5,600 | l.f. | 1.46 |
| 6 in. W. I. drain pipe and traffic posts | 750 | l.f. | 1.67 |
| 8 in. W. I. drain pipe | 430 | l.f. | 2.21 |
| Lighting system, lump sum | ....... | | 51,000 |
| Total | | | $2,056,400 |